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TRANSPORTATION RESEARCH RECORD

Journal of the Transportation Research Board, No. 2301

Operational Effects of Geometrics and Access Management 2012

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Foreword

The 2012 series of the *Transportation Research Record: Journal of the Transportation Research Board* consists of approximately 940 papers selected from more than 4,000 submissions after rigorous peer review. The peer review for each paper published in this volume was coordinated by the committee acknowledged at the end of the text; members of the reviewing committees for the papers in this volume are listed on page ii.

Additional information about the *Transportation Research Record: Journal of the Transportation Research Board* series and the peer review process appears on the inside back cover. TRB appreciates the interest shown by authors in offering their papers, and the Board looks forward to future submissions.

Note: Many of the photographs, figures, and tables in this volume have been converted from color to grayscale for printing. The electronic files of the papers, posted on the web at **www.TRB.org/TRROnline**, retain the color versions of photographs, figures, and tables as originally submitted for publication.

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To convert from the unit in the first column to the unit in the second column, multiply by the factor in the third column.

Customary Unit	*SI Unit*	*Factor*
Length		
inches	millimeters	25.4
inches	centimeters	2.54
feet	meters	0.305
yards	meters	0.914
miles	kilometers	1.61
Area		
square inches	square millimeters	645.1
square feet	square meters	0.093
square yards	square meters	0.836
acres	hectares	0.405
square miles	square kilometers	2.59
Volume		
gallons	liters	3.785
cubic feet	cubic meters	0.028
cubic yards	cubic meters	0.765
Mass		
ounces	grams	28.35
pounds	kilograms	0.454
short tons	megagrams	0.907
Illumination		
footcandles	lux	10.76
footlamberts	candelas per square meter	3.426
Force and Pressure or Stress		
poundforce	newtons	4.45
poundforce per square inch	kilopascals	6.89
Temperature		
To convert Fahrenheit temperature (°F) to Celsius temperature (°C), use the following formula: °C = (°F – 32)/1.8		

SI Unit	*Customary Unit*	*Factor*
Length		
millimeters	inches	0.039
centimeters	inches	0.394
meters	feet	3.281
meters	yards	1.094
kilometers	miles	0.621
Area		
square millimeters	square inches	0.00155
square meters	square feet	10.764
square meters	square yards	1.196
hectares	acres	2.471
square kilometers	square miles	0.386
Volume		
liters	gallons	0.264
cubic meters	cubic feet	35.314
cubic meters	cubic yards	1.308
Mass		
grams	ounces	0.035
kilograms	pounds	2.205
megagrams	short tons	1.102
Illumination		
lux	footcandles	0.093
candelas per square meter	footlamberts	0.292
Force and Pressure or Stress		
newtons	poundforce	0.225
kilopascals	poundforce per square inch	0.145
Temperature		
To convert Celsius temperature (°C) to Fahrenheit temperature (°F), use the following formula: °F = (°C × 1.8) + 32		

Abbreviations Used Without Definitions

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACRP	Airport Cooperative Research Program
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials (known by abbreviation only)
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ISO	International Organization for Standardization
ITE	Institute of Transportation Engineers
NASA	National Aeronautics and Space Administration
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
RITA	Research and Innovative Technology Administration
SAE	Society of Automotive Engineers
SHRP	Strategic Highway Research Program
TCRP	Transit Cooperative Research Program
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Motor Vehicle Speeds

Recommendations for Urban Sustainability

John N. Ivan, Thomas Jonsson, and Attila Borsos

This paper explores how vehicle speeds are related to equitable, environmental, and economic sustainability of urban areas. This relationship is manifested primarily through associations between vehicle speeds and road crash casualties, severity of pedestrian crashes, generation of harmful emissions, and relative desirability of neighboring land. Reported research findings describing these associations are presented and discussed. Reported experiences with implementing various methods of influencing vehicle speeds, including automated enforcement, self-explaining roads, and in-vehicle systems, are presented and discussed. To support increased sustainability of urban areas, the following steps are recommended: (*a*) speed limits should be set to limit casualty risk and not to accommodate driver choices, (*b*) roadways in developed areas should be designed with 10-ft lanes and on-street parking and sidewalks, and (*c*) vehicle speeds in downtown and residential areas should be kept below 25 mph (preferably 20 mph). The paper also identifies gaps in knowledge about speed and sustainability.

Sustainability has been a hot topic recently, especially in the context of the environment, urban development, and transportation. Although preservation of the planet and its resources for use by future generations is an important element of sustainability, maintaining quality of life, or livability, for current residents is also important to keeping cities sustainable. With the recent economic crises sweeping the globe, maintaining the ability to financially support societal values is also critical. The planning, design, and operation of urban development and transportation are inextricably linked to sustainability. According to Woodcock, "Sustainability in urban design seeks to establish a sense of place by enhancing the public domain. This may be evidenced by an effective public transport network, safe streets, equity of access to retail and services as well as traditional commons such as parks" (*1*). Woodcock identifies one of the key metrics for evaluating a sustainable transportation design as a "reduced injury rate." Similarly, the World Business Council for Sustainable Development, in its Sustainable Mobility Project, defines deaths and serious injuries by any travel mode as an indicator of sustainability (*2*). Vehicle travel speeds are directly and indirectly related to these metrics.

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For example, the extent to which existing or proposed road network and urban development patterns contribute to casualties in road crashes is an important element of the equitable (or social) sustainability of an area. Vehicle speed is an important indicator for crash casualties; therefore, equitable sustainability can be compromised in neighborhoods with vehicle speeds above acceptable levels.

In addition, when residents do not feel safe walking or bicycling in an area because of the high speeds on the roads, the attractiveness of the modes of walking and bicycling is reduced, as is the public's willingness to use those modes. This public unwillingness can make it more difficult to increase use of such modes to achieve environmental sustainability by reducing the generation of carbon and other harmful emissions. Higher vehicle speeds are associated with greater consumption of fuel and thus greater tail pipe emissions, which also work against environmental sustainability.

Neotraditional and traditional neighborhoods are proving to be highly desirable housing locations in the United States, mostly because of the walkability of the neighborhoods. Consequently, economic sustainability (especially property values) of newly developed and existing neighborhoods depends on keeping traffic speeds at levels low enough so that people feel safe walking and to find the neighborhoods livable and attractive, while still allowing adequate access to and egress from the area.

The objective of this paper is to explore how motor vehicle speeds are related to urban sustainability. The authors specifically examine research on the following questions related to vehicle speeds:

1. How are vehicle speeds associated with road casualties?
2. How do vehicle speeds affect vulnerable road users (e.g., pedestrians and bicyclists)?
3. How are vehicle speeds associated with harmful tail pipe emissions?
4. How are vehicle speeds associated with economic sustainability?
5. How can driver choice of speed be influenced by enforcement, environment (roadway and roadside design), and electronics?

The remaining sections of this paper address each of these questions; the final section summarizes the findings and presents recommendations for achieving urban sustainability through vehicle speeds.

VEHICLE SPEEDS AND ROAD CASUALTIES

Speed and Road Safety

There is a strong conceptual link between travel speed and road safety. Higher driving speeds allow drivers less time to process information and react [see Figure 1 (*3*)]. Furthermore, higher driving speeds

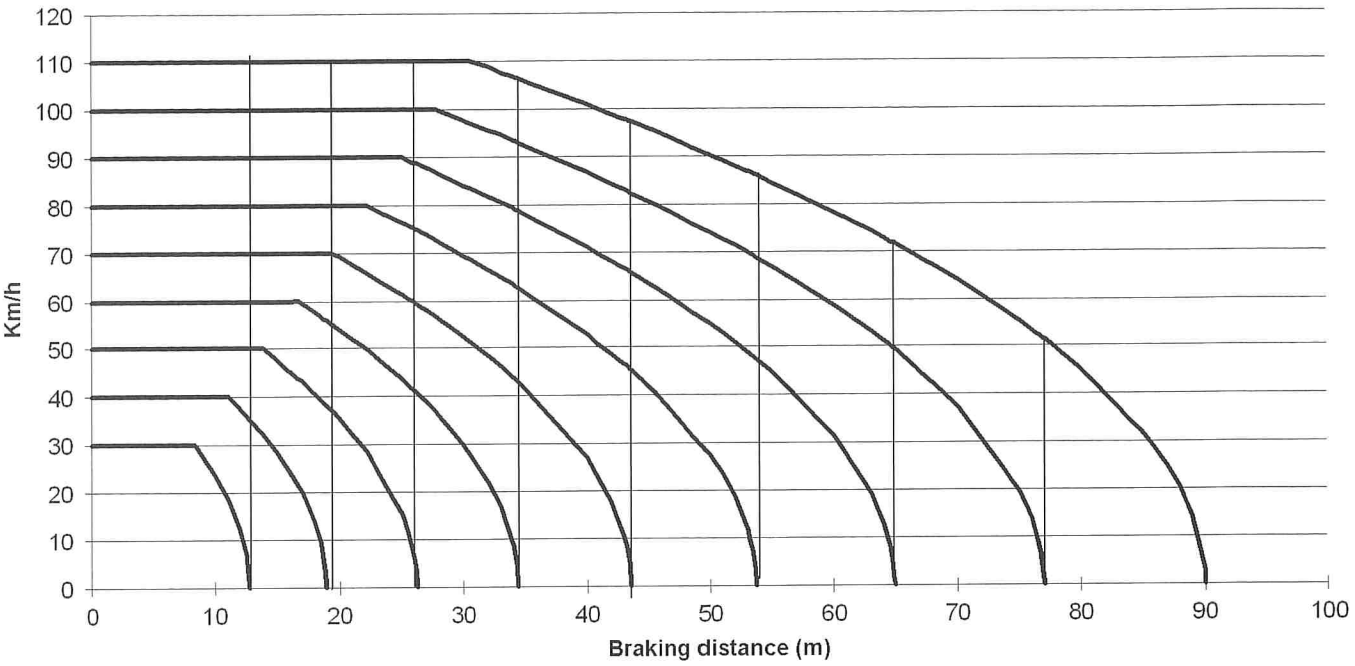

FIGURE 1 Stopping distance in relation to driving speed, calculated with deceleration = 0.8 *g* and reaction time = 1 s (good conditions) (*3*).

usually result in higher collision speeds and an attendant increase in crash severity and crash fatality rates.

Rosén et al. summarized and analyzed much of the work done before 2009 into the connection between impact speed and injury severity of pedestrians hit by a car (*4*). Early studies have suffered from biases toward severe outcomes because sampling issues were not taken into account. Results from the most reliable studies are presented in Figure 2. Above 75 km/h (about 46.6 mph), the pedestrian is more than 50% likely to be killed. The impact speed must be below 30 km/h (about 18.6 mph) for a negligible probability of the pedestrian being killed.

Hauer presented an overview of studies on speed and safety and noted several reasons why it is hard to identify this complex relationship (*5*). One reason is that crashes at higher speeds lead to higher severities. They are thus more likely to be reported than crashes at lower speeds. Another reason is that for a study to focus on the known speeds of vehicles involved in crashes, the precrash speeds must be estimated because it is impossible to observe them without a special study design. Consequently, the accuracy of the resulting analysis depends on the quality of the precrash speed estimation. Finally, for crashes involving turning vehicles, the speeds at the time of impact will be lower because of the turning maneuver, and this lower speed would not be reflected in the distribution of speeds of general traffic on the road.

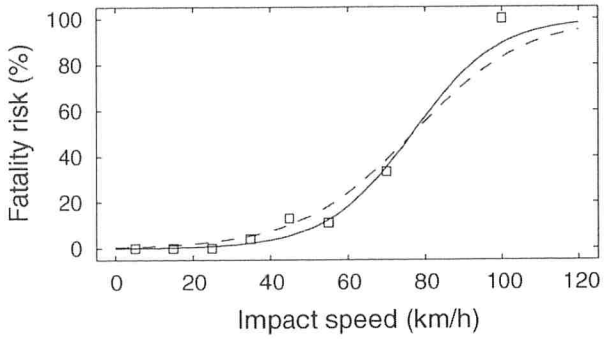

FIGURE 2 Fatality risk for pedestrians, according to impact speed of car (*4*).

Another reason for the difficulty in identifying the relationship between vehicle speeds and safety is the correlation between speed and just about every variable describing the road environment. In any cross-sectional comparison of crash rates between different road types, the high-speed roads often have the lowest crash rates (*6, 7*). The roads with the highest speeds, freeways, have the lowest crash rates, not because of the high speeds but because of the safe design features that cue the drivers to high speeds.

To explore this complicated relationship between vehicle speeds and safety, the next several sections address different aspects of the relationship:

- Observed vehicle speeds and crash incidence,
- Speed limit changes and crash incidence, and
- Observed speed variance and crash risk.

Observed Vehicle Speeds and Crash Incidence

Stout and Souleyrette conducted a case–control study of automatic traffic recorder speeds and crash incidence (*8*). They compared speeds collected for 1 h at the time of a known crash with speeds in the same hour 1 week earlier. The mean of the vehicle speeds was significantly higher during the case hours (with a crash) relative to the control hours (without a crash) on freeways and two-lane roads. The variance was significantly different between the case and control hours on freeways.

Davis et al. performed a case–control study of run-off-road crashes considering precrash speeds of individual vehicles (*9*). The objective was to investigate the frequently reported U-shape relationship between speed and the probability of being in a crash. This study addressed several problems of previous efforts: (*a*) case speeds have usually included turning vehicles, whereas control speeds have been observed in free-flow conditions; (*b*) multivehicle collisions are more likely in congested traffic and thus occur at lower speeds; and (*c*) most studies aggregate all collision types, but different collision types are more likely to occur at different speeds. Davis et al. addressed these issues using Bayesian methods to account for uncertainty in the observation of the case speeds. They attempted to fit linear and quadratic relationships between speed and crash incidence and found that both fit equally well. The quadratic function monotonically increased over the entire range of observed speeds, as did the linear function. Their conclusion was that if there is an increase in risk at lower speeds, it is for particular types of collisions and collision scenarios, such as freeway congestion, rather than cases in which drivers choose lower speeds because of roadway and roadside design settings. They also noted that higher speeds do not necessarily result in crashes because many control speeds were higher than the crash speeds; the point is that the crashes occurred more frequently at higher observed vehicle speeds.

Speed Limit Changes and Crash Incidence

Nilsson presented a model for the connection between a change in speed limit and crash occurrence as well as severity (*10*). The model was developed from an analysis of the safety outcomes from speed limit changes in Sweden over a couple of decades. The findings are that the number of crashes expected on a specific traffic facility is proportional to the speed (limit or running speed) to the power of an exponent and that the exponent differs between severities and is higher for more severe crashes. Nilsson proposed the exponent 2 for injury crashes, 3 for severe injury crashes, and 4 for fatal crashes.

As an example, the equation for fatal accidents is

$$\frac{\text{Fatalaccidentsafter}}{\text{Fatalaccidentsbefore}} = \left(\frac{\text{Speedafter}}{\text{Speedbefore}}\right)^4 \quad (1)$$

Elvik et al. revisited the Nilsson model and recommended best estimates for a modified version of the power model (see Table 1) (*11*).

TABLE 1 Speed and Road Accidents: Evaluation of Power Model (*11*)

Accident or Injury Severity	Exponent	Interval
Fatalities	4.5	4.1–4.9
Seriously injured road user	3.0	2.2–3.8
Slightly injured road user	1.5	1.0–2.0
All injured road users (severity not stated)	2.7	0.9–4.5
Fatal accidents	3.6	2.4–4.8
Serious injury accidents	2.4	1.1–3.7
Slight injury accidents	1.2	0.1–2.3
All injury accidents (severity not stated)	2.0	1.3–2.7
Property damage-only accidents	1.0	0.2–1.8

Friedman et al. applied time series modeling to monthly fatalities in Israel for 6 years before and after the speed limit on interurban roads was raised from 90 to 100 km/h (*12*). In the 6 years before, fatalities declined steadily; after the speed limit was raised, fatalities jumped in spite of countermeasures, including road improvements and new laws requiring use of rear seat belts. Fatalities increased on both interurban and urban roads, indicating a spillover effect: 12.7% of the deaths on interurban roads and 8.3% on urban roads (10.6% overall) were attributable to the change in speed limit.

Souleyrette et al. studied the effect of raising the speed limit from 65 mph to 70 mph on rural Interstates in Iowa (*13*). They found that the mean and 85th percentile speeds increased by 2 mph, so that the proportion of drivers exceeding the speed limit by 10 mph or more was reduced from 20% to 8%. They found that the average rate of fatal crashes per year increased by 21.8% in the 2½ years following the change compared with the 14½ years before the change. However, the average rate of serious crashes (fatal and major injury) per year actually decreased by 12.1% for the same comparison. It is possible that this change represents a shift in severity rather than an increase or decrease in crashes. This change might be further illustrated by noting that the actual number of fatal crashes per year increased on average from 19.2 to 25.2, that is, 31.3%, whereas the actual number of serious crashes (fatal and major injury) increased from 78.8 to 90.8, or 15.2%.

Holló et al. (*14*) investigated the impacts of changing the speed limits in Hungary. The aim of this study was twofold; the authors studied the safety impacts inside (speed limit lowered from 60 to 50 km/h in 1993) as well as outside (speed limit raised by 10 km/h in all road categories in 2001) built-up areas. In the former case, a control group test was performed that showed that the number of fatalities decreased by 18.2% in the following 3-year period. In the latter case, the author applied time series modeling and proved that after 2001, the number of fatalities increased significantly.

Observed Speed Variance and Crash Risk

Lave (*15*) argued that it is not high speeds that increase the incidence of crashes but rather the high variation among the speeds of vehicles on a road, appealing to the notion that when all vehicles travel at about the same speed there is a lower risk of collisions occurring, irrespective of what that speed is. Both Lave and Hauer provide several examples demonstrating this relationship, with Hauer's being the more compelling (*16*). One of the extrapolations from this finding is the suggestion that a driver can reduce his crash risk by driving at the average speed of the other vehicles on a road, that is, reducing the difference between his speed and that of the other vehicles.

Davis argued that this association between higher speed variance and crash incidence is an "ecological fallacy," a phenomenon in which "a relationship observed between aggregated variables is necessarily attributed to the basic entities over which the aggregation was done" (*17*). In other words, Davis argued that this relationship is an artifact of the aggregation of crash counts on individual roads. In particular, he pointed to the need to "distinguish between individual and aggregated measures of risk" and that "inferring a relationship between individual risk and speed dispersion from an observed relation between aggregated risk and speed dispersion is an ecological fallacy." Consequently, Davis did not dispute that aggregate crash risks are higher on roads with higher speed variance; he disputed that this observation can be translated into arguing that it is the higher speed variance that causes the increased crash risk.

VEHICLE SPEEDS AND VULNERABLE ROAD USERS

One important approach for increasing the environmental sustainability of urban areas is to reduce the demand for motorized vehicle travel through promotion of travel on foot and by bicycle. High vehicle speeds in a traffic environment deter people from walking. Alvén and Håkman interviewed bus travelers and residents (potential bus travelers) about their mode choice (*18*). They found that one of the main factors deterring travelers from walking and bicycling was a perceived lack of safety because of high volumes and speeds on the roads near the bus stop. This section reviews research on how vehicle speeds may affect the safety of these other modes and thus their attractiveness to travelers.

Davis et al. studied the effect of vehicle volumes and speed on the severity of pedestrian and motor vehicle crashes and conflicts in the Twin Cities (Minnesota) area (*19*). They noted that many jurisdictions in the United States use the "85th percentile rule" for setting speed limits. That is, they set the speed limit at a level that is exceeded by no more than 15% of the drivers. The justification of this rule is to avoid enforcement problems and the notion, derived from microeconomics, that drivers will consistently choose their speeds to balance their desire to reach their destinations quickly with the risk of a collision. They further note, however, that this microeconomic model breaks down in the case of pedestrian and motor vehicle collisions. In this case, the pedestrian usually incurs nearly all of the personal costs of a collision. The motorist's costs are usually limited to minor property damage that is often covered by insurance, while the pedestrian experiences a physical injury resulting in personal pain and suffering and the loss of time and mobility. Consequently, the motorist will choose a higher travel speed than the pedestrian would like. Davis et al. noted several studies confirming this finding. Their own study observed speeds and traffic volumes on 25 residential streets and found that the probability of a crash involving a pedestrian and a motor vehicle increased with the traffic volume but not with the traffic speeds, whereas the probability of such a crash being severe increased with the traffic speeds but not with the volume. These findings support reduction of vehicle speeds in locations where it is desired to increase the use of walking as a travel mode.

An FHWA report described development of intersection safety indices for bicycles and pedestrians (*20*). The indices are ratings between 1 and 6, with 1 being conditions that are safer for bicycling or walking and 6 being conditions that are completely unsafe. These ratings were calibrated with a group of safety experts. In the index estimated for pedestrians, the 85th percentile speed of traffic increased the index by 0.018 for each mile per hour. In the index for bicyclists traveling through the intersection, a main street speed limit over 35 mph increased the index by 0.815 compared with roads with lower speed limits.

VEHICLE SPEEDS AND TAIL PIPE EMISSIONS

One of the most obvious aspects of traffic related to sustainability is vehicle emissions. Vehicles emit chemicals that affect the health of those who dwell in the vicinity of the road, cause acidification and overfertilization, and add to global warming. The amount of vehicle emissions depends on speed. Aggressive driving, for instance, can increase fuel consumption by 40% and emissions by up to eight times (*21*). How emissions vary with speed depends on the type of emission.

The emission levels of nitrogen oxide depend mainly on the level of speed. The higher the speed, the more nitrogen oxide vehicles emit, but increased levels of acceleration will also increase the nitrogen oxide emission rate (*22, 23*). The emission levels of carbon monoxide and carbon dioxide do not depend as heavily on speed, although they can increase slightly with increased speed. For carbon monoxide emissions, acceleration drastically increases the emission levels (*24*), compared with cruising at constant speed.

Smidfelt (*25*) studied street networks and features that affect emission levels of carbon monoxide, hydrocarbons, and nitrogen oxide. Humps and intersections are generally considered to increase emission levels, but the study found that if the system effects in a wider street network are studied, instead of just the effects close to the hump or intersection, then smoother speed profiles at lower speeds can generate lower emissions.

Jackson described an experiment in which test runs with an instrumented passenger car were observed to learn how to more accurately model the generation of ultrafine particulate emissions as a function of vehicle dynamics (*26*). The vehicle was instrumented with an accelerometer, Global Positioning System receiver, onboard diagnostic system scan tool, and a particulate emissions collector. Jackson found that high emissions of ultrafine particulates occurred most frequently where vehicles accelerate rapidly (greater than 3 mph/s) or must climb steep grades. This is a logical finding; it suggests that for environmental sustainability (with respect to air quality), it is more effective to reduce the need for vehicles to change speed rather than to force them to repeatedly reduce speeds and then accelerate. This finding suggests replacing traffic signals and stop-controlled intersections with modern roundabouts and designing roads to carry vehicles at a constant speed rather than varying speeds.

VEHICLE SPEEDS AND ECONOMIC SUSTAINABILITY

The direct effect of vehicle speeds on economic sustainability has not been extensively researched, although it can be assumed that there are positive and negative effects. A positive effect would be the possibility to transport goods and people faster from one place to another, and a negative effect would be reduced attraction to live along the corridor of a high-speed road because of noise and perceived unsafe conditions. These effects have been investigated indirectly by several studies that evaluated property values along roads and streets where the standard has been improved (and logically the speed increased).

Vadali and Sohn studied the effect of the redesign of an expressway in Dallas, Texas, on property values in the expressway corridor (*27*). The study followed price changes during the project, but was reported before effects after the project could be estimated. The conclusions were that properties closest to the expressway saw the largest effects, negative in the beginning of the project, but rebounding to positive toward the finalization of the project. Ten Siethoff and Kockelman studied similar effects of improvement to the US-183 corridor through Austin, Texas (*28*). The findings were similar to those of Vadali and Sohn, with negative impacts during construction, but increased property values when finished, and the change in value decreasing with distance from the project. Carey and Semmens (*29*) studied the effect of freeway development on nearby land use

and property values. The findings were ambiguous, with increase in property values for some types of land use (multifamily housing and commercial property) and negative for some (single-family housing), indicating that single-family housing is more sensitive to disturbance. The most interesting result, however, was that the most negative impact on property values was not induced by the freeway per se, but by the increased traffic it induced on the roads in the area. A cross-sectional study of arterial corridors in Chicago, Illinois, (*30*) concluded that the more traffic there is on an arterial, the lower the property values.

Iacono and Levinson studied home sales in Minnesota and the effect of freeway proximity on property values (*31*). They found that the proximity to the freeway itself reduced property values, but proximity to its access points increased property values. This finding further shows that disturbances from high-speed road traffic and the benefit of improved accessibility work in opposite ways on economic values.

ACHIEVING SUSTAINABLE VEHICLE SPEEDS

Compliance with the posted speed limit is an important element of sustainability. Getting drivers to comply with the safe, sustainable speed on a street is a critical part of achieving sustainability. There are three ways to achieve speed compliance: enforcement, design of the roadway and roadside, and in-vehicle systems. Experience in these areas is discussed in this section of the paper.

Speed Compliance Through Enforcement

For this paper, enforcement includes law enforcement programs as well as programs that adjust how speed limits are chosen.

Rodier et al. reviewed automatic speed enforcement programs in the United States (*32*). Most of these programs were implemented on residential streets, with two exceptions: one program in the District of Columbia on a high-speed arterial and another in Arizona on an urban freeway. The various programs resulted in a 2% to 15% reduction in speeds and a 9% to 50% reduction in crashes. Few of the programs were profitable, and most required a subsidy. Public suspicion of a profit motive turns opinion against the program. This suspicion can be averted by pledging to apply proceeds from the program to safety improvement funds. The biggest impediment is with implementation because such programs require enabling legislation at the state level and frequently face challenges of constitutionality. There are also issues with vehicle owner versus driver liability that must be addressed in the legislation.

Aarts et al. described SaCredSpeed, a program of the Dutch Institute for Road Safety Research to set policy for determining speed limits (*33*). In this program, speed limits are set according to design, traffic mix, and location of the facility, with special consideration of how pedestrians and bicyclists are accommodated. For example, without physical separation between motorized and nonmotorized traffic, a maximum speed of 30 km/h is indicated. Speeds of 80 km/h or more are permitted only with separation of driving directions and a forgiving roadside. This program identifies "accelerators" and "decelerators," factors that increase or decrease the expected reasonable or credible speed perceived by drivers. It also discusses the role of enforcement, seeking to not require the police to enforce limits that the public deems to be inappropriately low, which reduces respect for the law enforcement.

Similar guidelines for setting speed limits according to traffic mix were developed in Sweden in the late 1990s, following the adoption of a bill known as "Vision Zero and the traffic safe society." Guidelines for urban areas were developed that state that the speed limit should not exceed 30 km/h on streets and at crossing points where vulnerable road users are found. The speed limit should not exceed 50 km/h where angle collisions between motor vehicles are possible. Where head-to-head collisions are possible, the speed limit should not exceed 70 km/h. These limits were set on the basis of the collision forces that the combination of the human body and the protection of the vehicle can sustain without considerable risk of a fatal outcome (*34*). After the introduction of new speed limits in 2008, additional guidelines were developed that detail how the speed limit should be set according to the design of the road (e.g., separation of vulnerable road users, presence of median barriers, and the like).

Hydén et al. studied the change in speed during a test of new speed limits in 12 Swedish cities (*35*). On most of the streets, the speed limits were lowered, from 70 to 60 km/h, and from 50 to either 40 or 30 km/h, but for some streets the speed limits were increased from 50 to 60 km/h or from 30 to 40 km/h. On average, a decrease of the speed limit resulted in a decrease of the actual speeds by 2 to 3 km/h, and an increase of the speed limit resulted in an increase of the actual speeds by 0 to 1 km/h. The changes of actual speeds were found to depend on the speed before the change. In most cases, the speed limit was changed to better agree with the actual speeds.

Speed Compliance Through Roadway and Roadside Design

The second approach to achieving desired vehicle speeds is by roadway and roadside design. This area has seen much work since the early 1990s, beginning in the Netherlands with the concept of "self-explaining roads" (*36*). The idea of self-explaining roads is that the road design should give the driver the right expectations and elicit the correct behavior. An ideal self-explaining road should, in theory, not even need a speed limit because it should make it evident to the driver what the correct speed would be. In fact, according to an FHWA report, simply reducing the posted speed limit on a roadway, even by as much as 15 mph, had no impact on mean travel speeds and did not reduce accident rates (*37*). Therefore, achieving sustainable, safe speeds on urban and suburban roads depends on creating the right roadway and roadside environment to guide drivers to choose a speed that is safe for them and for other road users.

Hansen et al. explored the relationship between roadway and roadside characteristics and observed vehicle free-flow speeds (*38*). They observed free-flow speeds using handheld radar guns at 272 locations on two-lane roads in Connecticut. Roadway characteristics that were collected included lane and shoulder width, presence of curbs, and edge delineation types. Roadside characteristics included presence of on-street parking, presence of sidewalks, size of the building setback, driveway density, and land use types. Analysis of variance was employed to identify significant variables in predicting mean speeds. The researchers observed higher speeds on roads with wide shoulders, large building setbacks, and no sidewalks or on-street parking and residential locations. They found

the difference in mean speeds to be as high as 10 mph between roads with different combinations of roadway and roadside design features. On the basis of their findings, they recommended that roads in settled areas be designed to have shoulders of no more than 2 ft in width and to include sidewalks and on-street parking to encourage drivers to travel at slower speeds that will promote greater safety for nondrivers.

In a synthesis of traffic calming on rural roads, Gorrill summarized findings from various studies on physical and operational measures to reduce speeds on through roads and minor roads (*39*). The most effective treatments were speed tables, although they also elicited the most complaints from residents. Otherwise, center islands that provided longitudinal narrowing worked the best.

Weller et al. (*40*) conducted a psychological laboratory study using photographs of rural roads to categorize roads by the speeds that drivers chose. They found that drivers' speeds tend to be higher when comfort or monotony is considered to be high.

Speed Compliance Through In-Vehicle Systems

Cunto et al. studied truck speed limiters, with simulation using VISSIM (*41*). Safety was represented by a crash potential index. The researchers simulated scenarios with trucks having speed limiters at 105 km/h. They found that the introduction of speed limiters would improve safety in uncongested conditions, especially on simple freeway segments. The safety gains were reduced as traffic volumes increased, and actually reverse in congested conditions in the vicinity of on-ramps and off-ramps. The issue in those situations was increased vehicle interactions and the likelihood of being unable to avoid a collision.

Extensive experiments with speed limiters in cars were carried out in Sweden from 1999 through 2002 as part of the Intelligent Speed Adaptation (ISA) project. The experiments were conducted in four cities and with different systems with different levels of feedback or guidance for in-vehicle information about the speed limit, warnings about speeding through sound or light, and actual limitations of the possible driving speed (*42*).

The main findings follow:

- Road safety was increased without increasing travel time.
- If everyone had ISA technology, there could be 20% fewer road injuries in urban areas.
- ISA enjoyed high acceptance, and after the trial most test drivers were of the opinion that ISA should be compulsory in urban areas.
- ISA vehicles were found to have a positive influence on surrounding traffic.
- Minor differences occurred between the systems, with an average speed reduction of 3 to 4 km/h on stretches between intersections.
- The systems must be improved to become more attractive.

Results from the ISA experiments with actual limitation of the speed were further studied by Hjälmdahl (*43*). The system used an active accelerator pedal that gave a much increased back pressure when the speed limit was reached. The conclusions were that drivers using the system were both driving with a reduced mean speed as well as reduced speed variance. The author estimated an injury reduction of up to 25% if all vehicles were equipped with the system.

CONCLUSIONS

This paper has demonstrated that high vehicle speeds can exasperate initiatives to improve urban sustainability in several ways:

1. High vehicle speeds increase the likelihood of pedestrians being killed when struck by a motor vehicle and increase the general severity level of any crash, reducing equitable sustainability.
2. High vehicle speeds increase the generation of nitrogen oxide emissions, reducing environmental sustainability.
3. High vehicle speeds have mixed effects for economic sustainability. They reduce sustainability by negatively affecting the attractiveness and marketability of residential neighborhoods close to high-speed roads, but access to high-speed roads also increases economic sustainability in urban areas because these roads are critical for movement of people and goods.

Furthermore, the paper has identified certain critical speed thresholds that are useful for defining what a sustainable speed for various urban environments is. Examples follow:

1. Speeds below 30 km/h (18.6 mph) result in negligible risk of pedestrian fatalities.
2. At speeds above 75 km/h (46.5 mph), a pedestrian is more than 50% likely to be killed in a collision with a motor vehicle.
3. Reducing vehicle speed changes has greater benefits for environmental sustainability.

Finally, the paper has shown that, apart from traditional police enforcement, it is possible to influence driver speeds to sustainable levels.

1. Automated enforcement can reduce speeds up to 15% and crashes up to 50%.
2. Setting speed limits according to rational outcomes rather than by observed 85th percentile driver speeds can achieve speed limits set at levels appropriate to the land context and reduce crash casualties.
3. Drivers consistently take cues from roadway and roadside design elements about the appropriate speed to choose.
4. Drivers will accept technology to help them control their speeds at safer levels.

As a result of these findings, the authors recommend that urban road authorities consider managing vehicle speeds in their jurisdictions as a critical element of their plans to achieve sustainability. In particular, the authors recommend the following:

1. Consider guidelines such as the Dutch SaCredSpeed or the Swedish Vision Zero for setting speed limits in urban areas.
2. Consider limiting lane and shoulder widths and including sidewalks and on-street parking in settled areas, especially in residential zones.
3. Speeds in areas with human development, especially pedestrian activity, should be kept lower than 25 mph, preferably 20 mph, to eliminate pedestrian fatalities.
4. Roads should only be designed with speeds greater than 30 mph outside urbanized areas or in urbanized areas where pedestrian and bicycle facilities are physically separated from motor vehicles.

In urbanized areas it is important to distinguish between streets providing direct access to human activity and land development and those that do not provide direct access to human activity and land development. The former type of street should focus on access and have lower speeds. The latter type of street could have higher speeds to serve the mobility function, provided pedestrians are adequately separated from motor vehicles. There should be a balanced network of both types of streets in the city, with a limited number of the higher-speed streets. At the same time, the authors' review of existing research has revealed the following shortcomings in the extant knowledge about speed and sustainability:

1. It is extremely difficult to isolate the association between speed and crash incidence. Some researchers, such as Davis et al. (*9*), have made great strides using causal analysis and other microscopic approaches to account for precrash speeds of vehicles that were actually in collisions as compared with the running speeds of vehicles not in collisions. However, none of the existing research is able to cite specific speed thresholds associated with reduced crash rates.
2. Vehicle emissions are related more to acceleration than to speeds. Consequently, research is needed on how various speed, road design, and land development scenarios affect vehicle accelerations.

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The Operational Effects of Geometrics Committee peer-reviewed this paper.

Consideration of Sight Distance in Placement of Concrete Barriers on Horizontal Curves of Roads

Mohamed Sarhan and Yasser Hassan

Sight distance is an important criterion in highway geometric design. Many design elements, such as horizontal and vertical curvatures and offsets to lateral obstructions, must be checked so that drivers have sufficient sight distance to operate safely. One of the obstacles that can restrict sight distance is the roadside or median concrete barriers that are frequently placed along the road edges for safety. There are common types and dimensions for such barriers, such as New Jersey, F-shape, and constant-slope concrete barriers. Such barriers may cause a significant reduction in the available sight distance on horizontal curves, especially in the lane closest to the barriers. However, there is a clear lack of a reliable, accurate tool to determine the available sight distance when the sight distance is restricted by road barriers in three-dimensional combined alignments. This paper uses a software package previously developed by the authors to examine closely the trends of available stopping sight distance and the effects of the overlapping vertical alignment. The results confirm previous findings that the available sight distance depends on the type of the vertical alignment and the curvature of crest or sag vertical curves overlapping on the horizontal curve. In addition, it is shown that the recommendations in current design guides could lead to barrier placement that compromises the available sight distance. Charts were prepared as an easy-to-use tool to estimate the available stopping sight distance on horizontal curves when overlapping with a specific vertical alignment.

Roadside barriers are placed in the longitudinal direction of high-speed roadways to redirect errant vehicles and shield them from hitting obstacles along either side of the road (*1, 2*). Barriers may also be placed in the median area to prevent out-of-control vehicles in one direction from crossing to the other direction; in this case, they are called median barriers. This paper presents a study of the impact of roadside and median barriers on the available stopping sight distance (SSD) on horizontal curves.

Different types of barriers are being used worldwide. According to the AASHTO *Roadside Design Guide* (*1*), each barrier falls under one of three main categories according to its deflection characteristics. The categories are flexible, semirigid, and rigid. Flexible barriers are the most forgiving; much of the energy resulting from a car impact dissipates by the deflection of the barrier. Semirigid barriers are similar to flexible ones except that the posts are relatively stronger. Rigid barriers are most appropriate where no space is available to accommodate barrier deflection. Because of their limited deflection, rigid barriers are particularly advantageous when shallow impact angles are expected (*1*). The most common type of rigid barrier is the concrete barrier. Figure 1 shows examples of the use of temporary and permanent concrete barriers in the median and on the roadside in Ottawa, Ontario, Canada, and in Cairo, Egypt.

In general, all barrier types have been developed in shapes and dimensions to make striking the barrier a more forgiving crash than striking other obstacles and terrain features. In addition, there are several warrants listed in the *Roadside Design Guide* (*1*) for the use of roadside barriers. These warrants are all based on the premise that traffic barriers should reduce the severity of potential crashes. Many factors were also mentioned to influence the selection of the most appropriate traffic barrier. These factors include embankment height, side slope, roadside obstacles, roadside activities, conflict between vehicles and pedestrians, likelihood of run-off-the-road accidents, barrier performance, construction and maintenance cost, aesthetics, and field experience. For concrete barriers, there are different shapes, such as the New Jersey, F-shape, and constant-slope concrete barriers. In addition, these concrete barriers are available in two heights, 810 mm (32 in.) and 1,070 mm (42 in.). The barriers with these dimensions have been tested using the procedures and guidelines in NCHRP Report 350 (*3*). The selection of the barrier height depends on the danger level and type of vehicles that are likely to hit those barriers. Figure 2 shows a schematic cross section of the three 810-mm concrete barriers.

In addition to the barrier type and design, designers need to pay careful attention to the location (or placement) of the barriers. Intuitively, the presence of roadside or median barriers increases the probability of incidents of vehicles hitting an object. Yet the barriers are used on the premise that they will reduce the severity of crashes because of the presence of a more dangerous object beyond the proposed location of the barrier. Therefore, and to give drivers a chance to regain control of their vehicles, it is recommended to (*a*) place the barriers as far from the traveled way as possible and (*b*) provide a uniform clearance between the traffic and barriers (*1*). In addition, the AASHTO Green Book recommends placing the roadside barriers beyond the edge of the shoulder to ensure that the shoulder's full width is used by road users (*2*). Donnell and Mason (*4*) stated that barrier crash frequencies may decrease with the increase of median barrier offsets from the left edge of the traveled way. A similar trend of decreasing collision frequency with the increase of clear offset

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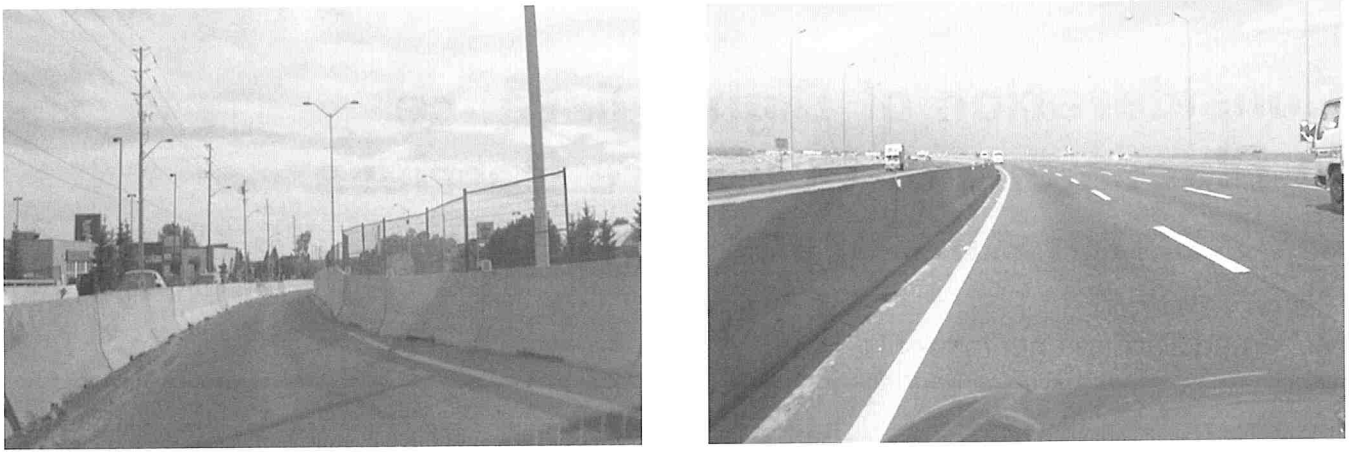


(a) (b)

FIGURE 1 Examples of concrete barriers in median and on roadside: (*a*) temporary barriers in Ottawa, Ontario, Canada, and (*b*) permanent barriers in Cairo, Egypt.

was also observed from the analysis of road collisions in European countries (*5*).

In all cases, the distance between the edge of the traveled way and roadside barriers is suggested to have a minimum value, as listed in Table 1, depending on the design speed (*1*). This distance is the minimum distance after which any roadside objects will not be perceived as obstacles; it is called the shy line offset. These shy line offsets, however, represent just one criterion in barrier placement that was suggested on the premise that drivers would neither reduce their speed nor change vehicle positions. Other criteria need to be considered by designers, including, for example, the chance that the driver of an errant vehicle should be given to regain control of the vehicle without hitting the barriers (*1*).

ROAD BARRIERS AND SIGHT DISTANCE

Another criterion that can be critical in the placement of barriers is their impact on the sight distance available to drivers. The Green Book recommends that designers give enough consideration to the impact of the barrier's design and placement on sight distance on horizontal curves (*2*). This effect would clearly decrease as the barrier's offset increases above the minimum shy line offset. This design is preferred whenever the right-of-way permits, which is the case in the placement of most permanent barriers. However, sight distance can be critically affected if the minimum shy line offsets in Table 1 end up as a standard offset in locations with restricted right-of-way, especially in highway construction areas. In addition, double-face concrete barriers are typically used as a cross-section element of several road types, such as freeways on earth embankment, and the median width of such a section is allowed to be as low as 3.0 m (*2*). Accounting for the barrier's width at the base, such a median width would allow for about 1.2 m lateral clearance on each side of the median concrete barrier. This design may result in relatively low values of available sight distance on horizontal curves, depending on the sharpness of such curves.

The only tool available in the Green Book to examine the barriers' effect on sight distance is the following formula, which relates available sight distance to lateral clearance on horizontal curves (*2*). In this formula, the lateral clearance is referred to as the horizontal

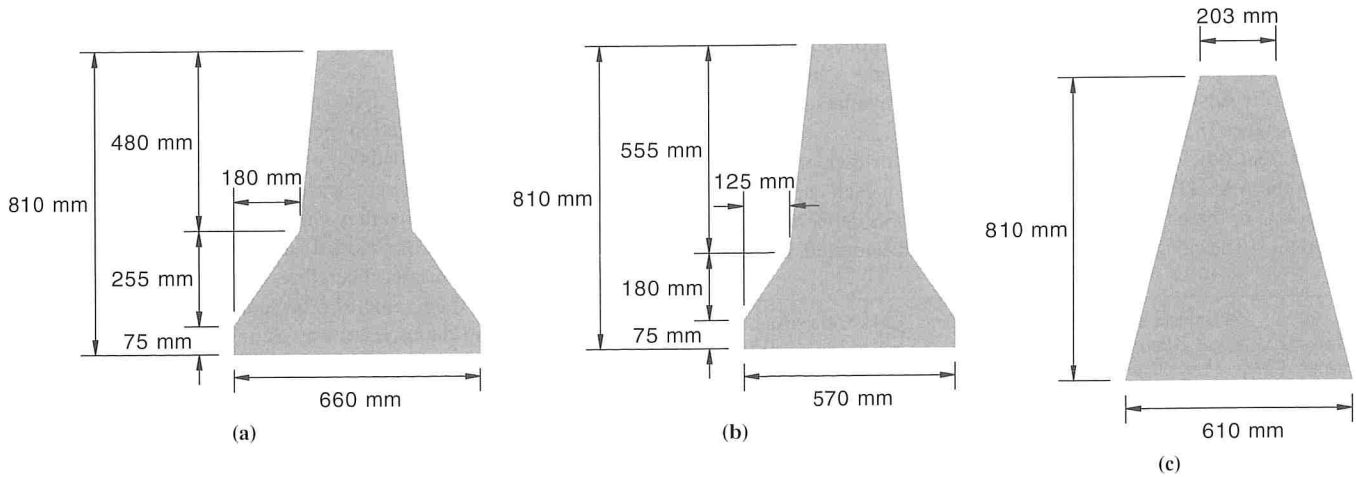


FIGURE 2 Common types of traffic concrete barriers: (*a*) New Jersey, (*b*) F shape, and (*c*) constant slope. [Source: Adapted from Figure C-6 of the *Roadside Design Guide* (*1*).]

TABLE 1 Recommended Shy Line Offset Values

Design Speed (km/h)	Lateral Offset (m)	Design Speed (km/h)	Lateral Offset (m)
50	1.1	100	2.4
60	1.4	110	2.8
70	1.7	120	3.2
80	2.0	130	3.7
90	2.2		

SOURCE: Adapted from Table 5.5 of the *Roadside Design Guide* (*1*).

sightline offset (HSO) and is measured from the centerline of the inner lane to the lateral obstruction.

$$\text{HSO} = R\left[1 - \cos\frac{28.65\,S}{R}\right] \quad (1)$$

where

S = available sight distance (m),
R = radius of curve (m), and
HSO = horizontal sightline offset (m).

As indicated in the Green Book (*2*), this equation applies only to circular curves longer than the sight distance where both the driver and the obstacle are on the circular part of the horizontal curve. In addition, the line of sight connects the driver's eye at a height h_1 to an object with a height h_2 that is located a distance S ahead along the centerline of the inner lane. When a vertical surface obstructs the sight line, HSO can be easily determined because it will have a constant value regardless of the heights h_1 and h_2. However, when the sight line is obstructed by a sloped surface (such as a cut slope) or an irregular surface (such as a concrete barrier), HSO will depend on the height at which the sight line touches the lateral obstruction.

The limitations of the formula above have been discussed further by many researchers [see, for example, work by Hassan et al. (*6, 7*)]. Among these limitations is that the formula cannot be applied on short horizontal curves or at the extremities of a long curve, nor can it consider the effects of the overlapping vertical alignment. Another limitation relevant to this study is the difficulty of accurately determining the height at which the sight line becomes tangent to irregular lateral obstructions, such as concrete barriers. This point of tangency is typically taken at the average height between h_1 and h_2. As indicated in previous work (*8*), this assumption is not accurate when the horizontal curve overlaps with a crest or sag vertical curve.

A search of the literature shows that few studies have addressed sight distance restrictions caused by roadside barriers. For example, Klam et al. (*9*) mentioned that shorter barriers, referred to as low-profile barriers, are being used in different locations, such as Texas and Florida, to enhance visibility at intersections. However, such barriers, with a height of 508 mm (20 in.) meet only the Test Level 2 criteria (70 km/h) of NCHRP Report 350 (*3*). To prepare such barriers for a higher test category, it was suggested that a stabilized rail be attached to the low-profile barrier. Richl and Sayed used the reliability analysis to evaluate the risk associated with insufficient available sight distance as a result of median barriers (*10*). It was mentioned that narrow medians are sometimes designed on tight horizontal curves to reduce the construction cost. A methodology was presented to calculate the probability of noncompliance that describes the associated risk of having a driver requiring a sight distance greater than available to make a safe stop. However, the paper relied on the closed-form equation for available sight distance, and thus suffers the same shortcomings discussed above. Arndt et al. also addressed the provision of sight distance around roadside concrete barriers (*11*). It was mentioned that the provision of SSD using the design criteria listed in the current design guides would lead to wide shoulders, which was described as uneconomical. The authors presented less-conservative criteria for the SSD parameters, such as deceleration rate and object height, which were thought to be more realistic in such cases. In addition, proposed criteria and assumptions were presented for two cases: when the sight line over roadside concrete barriers is possible and when such a scenario is not possible.

STUDY OBJECTIVES

On the basis of the information presented in the previous sections, this paper presents a study to examine the impact of roadside and median barriers on the available SSD on horizontal curves when overlapping with different vertical alignments. A tool previously developed by the authors for sight distance analysis on combined alignments is presented first. This tool is then used to analyze the trends of sight distance restriction by road barriers and effect of the overlapping vertical alignment. Finally, examples of design aids are presented in the form of charts that can be easily used by designers to determine the available SSD on a specific alignment as restricted by the placed barriers. All sight distance analysis presented in this paper corresponds to the inner lane on the horizontal curve. This lane will be the closest lane to barriers and thus will have the lowest HSO and will represent the critical lane in the traveled way. The analysis presented in the paper is applicable to roadside concrete barriers on right-turn horizontal curves and to median barriers on left-turn curves. Both types of barriers will be referred to as road barriers.

SIGHT DISTANCE EVALUATION

Evaluation Tool

Because of the limitations of the AASHTO formula, an alternative numerical methodology has been developed for the analysis of available sight distance on complex horizontal and vertical alignments in two-dimensional or three-dimensional (3-D) environments (*12, 13*). This methodology was originally coded into a computer program using the QuickBasic programming language. More recently, the methodology was recoded using the Visual Basic.NET object-oriented programming language into a software package called Sight Distance Evaluation System (SDES) (*14, 15*).

The software has a set of friendly interfaces through which users can identify the horizontal alignment, vertical alignment, cross sections, and any horizontal or vertical obstructions. The user also has the opportunity to define a set of design parameters related to sight distance, such as the driver's eye height, object height, speed, perception and reaction time, friction coefficient, and acceleration rate. The software is able to produce two profiles for both required and available SSD. The analysis can be done with the deterministic approach (a single predefined value is used for each design parameter) or the probabilistic approach (a predefined distribution is used for each design parameter). With the probabilistic approach, a risk profile is

generated to show the probability of drivers whose available SSD is less than the required SSD.

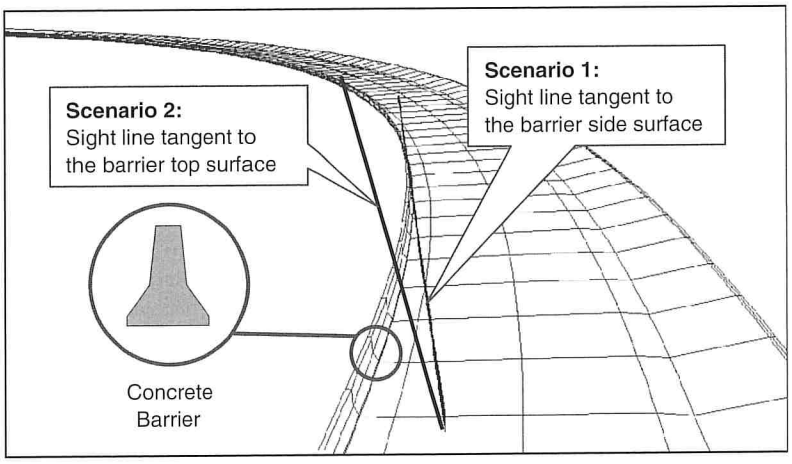
In this paper, SDES is used to determine the available SSD on a given road section in a 3-D environment. In this 3-D approach, the combined horizontal and vertical alignments of the road surface and side slopes are modeled into a finite element mesh as schematically shown in the subsequent section. The sight distance is then checked by testing the intersection between these elements and the sight line that connects the driver's eye to an object located at distance S from the driver's eye. The distance S is then increased until the sight line is tangent to any sight obstruction, which can be a lateral object or the road surface.

Modeling of Barriers

Similar to any lateral obstruction, the placement of road barriers at the inside of a horizontal curve will introduce a potential sight obstruction. For the purpose of this study, sight obstruction by barriers was modeled in SDES with additional elements at a specified lateral offset and with the dimensions corresponding to the New Jersey barrier shown in Figure 2. A schematic representation of the 3-D elements modeling the road surface and road barriers is shown in Figure 3*a*. This figure shows two possible scenarios for the obstruction of sight distance by road barriers. For the two scenarios below, the sight line is drawn from a driver's eye height, h_1, to an object height, h_2, with both points located at the centerline of the inner lane.

Scenario 1. The sight line is obstructed by the road barrier at a height lower than the height of the concrete barrier. This obstruction would take place at a point close to the midpoint of the sight line if both the driver's eye and object are on the horizontal curve. Because the object height in the calculation of SSD is normally lower than the barrier, this scenario is the most likely when the horizontal curve overlaps a crest vertical curve. In this case, the available SSD is the distance from the driver's eye to the object when the sight line is tangent to one of the barrier's sloped side surfaces.

Scenario 2. The sight line crosses over the barriers. This scenario can take place when the horizontal curve overlaps a sag vertical curve, which causes the midpoint of the sight line to be raised relative to the barriers. Therefore, the available SSD in this scenario is limited by the sight line being tangent to the top surface of the barrier.



(a)

(b)

FIGURE 3 Restriction of available SSD by median barriers: (*a*) possible scenarios and (*b*) numerical example for case of R = 500 m overlapping flat vertical grade.

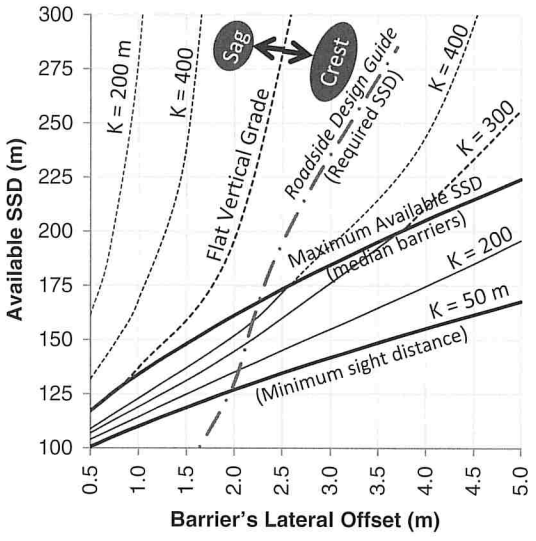
When no other features beyond the barriers (such as landscaping) obstruct the sight line, the available SSD in Scenario 2 will be greater than that in Scenario 1. For median barriers, however, the sight line should not be allowed to go beyond the edge of traveled way of the opposite direction so that the available SSD would not depend on whether there is a vehicle in the opposite direction. To simulate this scenario in SDES, a set of horizontal obstructions was placed along the edge of the opposite direction, as shown in Figure 3*b*. These obstructions were placed at a lateral offset, measured from the left edge of the median lane, equal to two times the barrier's lateral offset plus 660 mm, which is the barrier width. They were placed at a 10-m interval, and each one was formed of a rectangular element with width and height of 50 m and 100 m, respectively. Figure 3*b* also presents a numerical example of one SDES run in which a flat vertical grade overlaps a 500-m horizontal curve. As shown in the figure, the available SSD was checked at Station 200 and was 161.21 m because the line of sight was obstructed by one of the fabricated horizontal obstructions.

SSD TRENDS AND EFFECT OF VERTICAL ALIGNMENT

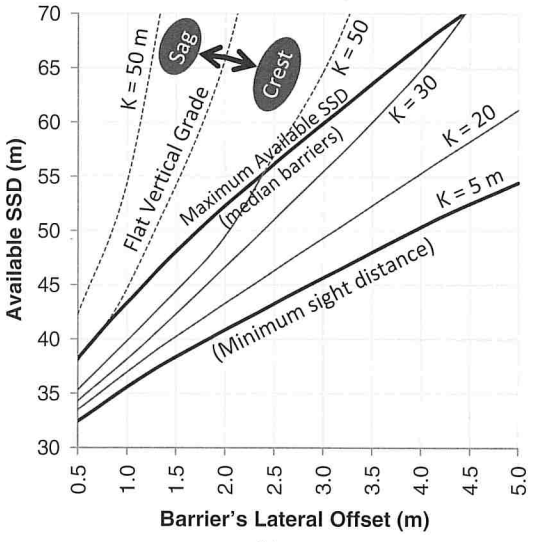
SDES was first used to closely examine the trends of available SSD restricted by road barriers using two horizontal curve radii: a relatively flat curve with 500-m radius and a relatively sharp curve with 50-m radius. The curve radii refer to the inner edge of the lane, which is closer to the roadside barriers. For example, if the median-side lane of a left-turn curve is to be evaluated, the radius of the left edge of this lane should be used (i.e., the edge between the median and the left lane). Several alignment combinations were created by overlapping the two horizontal curves with crest and sag vertical curves with different rates of curvature as well as a flat grade. The horizontal and vertical curves were set to fully overlap by starting and ending at same stations. In addition, both curves were taken as long as possible so that the highest possible value of available SSD was reached on the horizontal curve. In all these combinations, the road cross section was customized with a lane width of 3.6 m, superelevation rate of 4%, and New Jersey concrete barriers with the dimensions presented in Figure 2. The barriers were placed at variable lateral offsets as measured from the inner edge of the lane to the edge of the barrier. Each combination of horizontal and vertical alignments and barrier lateral offset represented an individual run in SDES. Finally, in accordance with the Green Book (*2*), h_1 and h_2 were set as 1.08 and 0.60 m, respectively, and were placed at the middle of the lane.

Flat Horizontal Curve

For the relatively flat horizontal curve ($R = 500$ m), the length of the horizontal and vertical curves was set to 600 m. Figure 4*a* shows the relationship between the barrier's lateral offset and available SSD corresponding to different cases of overlapping vertical alignment. As shown in the figure, the general trend indicates that, relative to the case of a flat vertical grade, the available SSD decreases as sharper crest vertical curves (lower K-values) overlap with the horizontal curve. In a 3-D environment, the sharper the overlapping crest vertical curve, the lower the middle portion of the sight line relative to the lateral obstruction. HSO and hence the available SSD will be reduced; refer to the cross section of the New Jersey concrete barriers in Figure 2.



(a)



(b)

FIGURE 4 Influence of barrier lateral clearance on available SSD on horizontal curves: (*a*) example of a flat curve, $R = 500$ m, and (*b*) example of a sharp curve, $R = 50$ m.

However, crest vertical curves sharper than those in the figure did not cause more significant reduction in the available SSD. A close investigation of this observation revealed that these cases correspond to the sight line tangent to the barrier at its lowest part. Because this part is vertical, HSO will not change with any further lowering of the sight line. However, sharp crest curves and large values of lateral offset will cause the available SSD to be restricted by the vertical curve; this restriction is outside the scope of this paper. Therefore, the curve in Figure 4*a* that represents the case of an overlapping crest vertical curve with a 50-m K-value was considered as the minimum available SSD for the horizontal curve with 500-m radius, and the results for sharper crest vertical curves are not shown in the figure.

As for the case of overlapping sag vertical curves in Figure 4*a* the available SSD is greater than that in the case of flat vertical

grade and increases as the sag vertical curve gets sharper. Again, the reason for this increase is the heightening of the middle portion of the sight line and hence the increase in HSO. For the cases of an overlapping flat vertical grade or sag vertical curve, as the lateral offset increases, the available SSD quickly reaches high values as the sight line passes over the barriers without obstruction. In such cases, if the horizontal curve is turning left, the sight line would cross over the median if it is clear from other obstacles, such as landscaping. As stated earlier, however, the sight line should not be allowed to cross over the traveled way of the opposite direction. Additional runs of SDES were therefore carried out for the horizontal curve overlapping with a flat grade and with a set of horizontal obstructions placed in the same way as described earlier. The resulting available SSD is shown in Figure 4*a* as the maximum available SSD for median barriers. The values of available SSD above this line represent values that are applicable only for roadside barriers, provided that the shoulder is clear from other sight obstructions.

Finally, Figure 4*a* shows a line for the required SSD corresponding to the minimum shy line offsets recommended by the *Roadside Design Guide* as shown in Table 1. In developing this line, the design speed corresponding to each value of the minimum shy line offset was used to calculate the required SSD using the following equation and rounding according to Exhibit 3-1 of the Green Book (*2*):

$$S_{req} = 0.278\,PV + 0.039\frac{V^2}{a} \quad (2)$$

where

S_{req} = required SSD (m),
P = perception–reaction time = 2.5 s,
V = design speed (km/h), and
a = deceleration rate = 3.4 m/s^2.

From the figure, it is obvious that minimum shy line offsets in the *Roadside Design Guide* (*1*) would not always secure enough SSD, and hence might increase the hazard to road users. For example, a minimum shy line offset of 2.4 m is recommended for a 100 km/h design speed, which corresponds to a required SSD of 185 m. However, such a lateral offset would result in an available SSD that is as low as 133 m if the horizontal curve is overlapping with a crest vertical curve with a 50-m K-value. Regardless of the overlapping vertical alignment, if the curve is turning left, the maximum available SSD is limited to around 170 m. To provide the same SSD of 185 m, a minimum offset of 3.0 m rather than 2.4 m might be needed.

Sharp Horizontal Curve

Similar to the flat horizontal curve, the values of the available SSD in some runs of the sharp horizontal curve (R = 50 m) are compiled in Figure 4*b*. In all these runs, the length of both horizontal and vertical curves was set as 150 m. The figure shows the same trends observed on the flat horizontal curve. That is, the available SSD (*a*) increases as the overlapping vertical alignment changes from crest curve to flat grade to sag curve, (*b*) decreases as the overlapping crest vertical curve becomes sharper, and (*c*) increases as the overlapping sag vertical curve becomes sharper. However, this last trend is not shown in the figure because the available SSD values quickly exceeded the limits of the chart. The figure also shows two curves for the minimum and maximum available SSD. Similar to the flat horizontal curve, the maximum available SSD is applicable for the case of median barriers on left-turn curves to ensure that the sight line would not cross over the traveled lanes of the opposite direction. However, unlike the case of the flat horizontal curve, the crest vertical curve with a K-value of 50 m no longer corresponds to the minimum available SSD. Thus, it can be concluded that the curvature of the crest vertical curve beyond which the available SSD would not decrease with sharper curves depends on the horizontal curve radius.

SAMPLE DESIGN AIDS

Now that the trends of available SSD when restricted by road barriers are established, there is a clear need to provide designers with easy-to-use tools to consider the effect of barrier placement on sight distance in 3-D environments. Such tools or design aids can be developed with SDES. As an example, Figure 5*a* compiles the

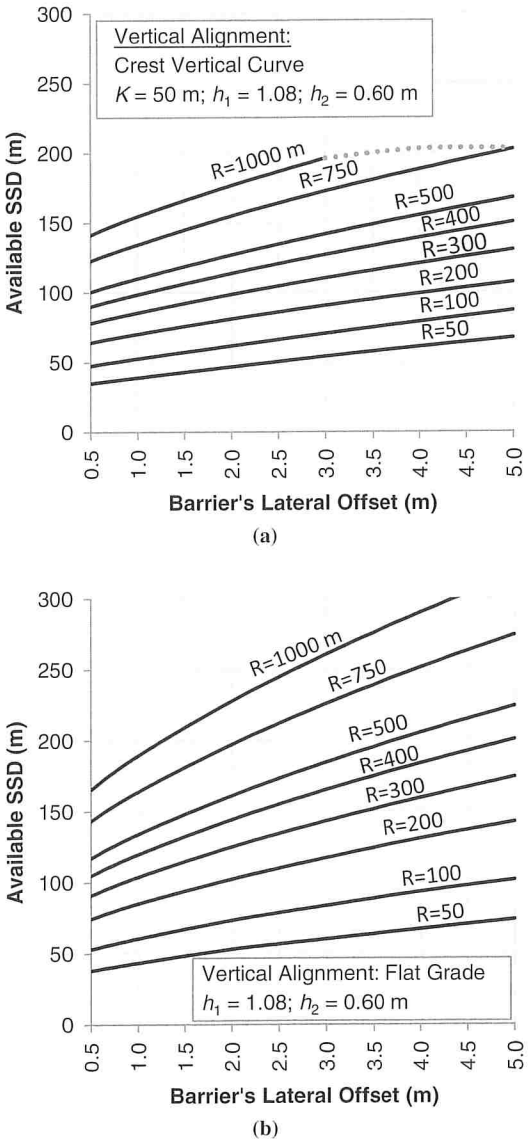


FIGURE 5 Available SSD on horizontal curves overlapping with (*a*) crest vertical curve and (*b*) flat vertical grade.

results of additional runs by SDES to determine the available SSD for different horizontal curve radii ($R = 50$ to 1,000 m) while overlapping with a specific crest vertical curve ($K = 50$ m). The cross section of the road was customized with a lane width of 3.6 m and superelevation rate of 4%. Figure 5*a* represents a tool that can be used to check the available SSD for a specific barrier's lateral offset. All SSD values shown in the figure correspond to obstruction by the road barrier and are applicable for both roadside and median barriers. Finally, the dotted part of the curve of $R = 1{,}000$ m and offset greater than 3.0 m corresponds to a case of the sight line being obstructed by the road surface rather than the roadside barrier. In this case, and because of the flat horizontal curve and large lateral offset, the available SSD is controlled by the relatively sharp crest vertical curve and is almost constant.

Another example is the chart shown in Figure 5*b* for the case of horizontal curves overlapping with a flat vertical grade. In the creation of this chart, lateral obstructions were also added at the edge of the opposite traveled way. This case corresponds to sight obstruction by median barriers and possible obstructions on the opposite direction on left-turn horizontal curves. The two figures indicate that the difference between the available SSD for each pair of values that correspond to a certain design scenario is considerable. For example, the case of $R = 500$ m and barrier lateral offset = 2 m yields available SSD of 126.92 and 161.21 m for $K_{crest} = 50$ m and flat vertical grade, respectively. In other words, the available SSD in such a case would fall in this range, depending on the vertical curvature. Charts similar to those in the two figures can be developed for horizontal curves overlapping with different vertical alignments.

CONCLUSIONS

Although designers are encouraged to keep a clear zone on the roadside to allow out-of-control vehicles to recover, such a design is not always feasible. Therefore, the general rule has been to place barriers along the side of the road or in the median if the risk of hitting an obstacle beyond the barrier exceeds the risk of hitting the barrier itself. Current practices indicate that concrete barriers with a height of 810 mm are used frequently on different road classifications as permanent road features or as temporary features in construction zones. The Green Book recommends that designers consider the impact of these barriers on sight distance. However, no tool is given to effectively and accurately accomplish this task. In this paper, available SSD on horizontal curves with roadside or median barriers was analyzed in 3-D combined alignments using software and the finite element technique. The need to consider the 3-D nature of roadways in design was noted as a strategic need by TRB committees (*16*).

The analysis in this paper showed that the minimum shy line offsets recommended in the AASHTO *Roadside Design Guide* can yield available SSDs lower than those required by drivers for safe stopping. The analysis also confirmed that the available SSD on a horizontal curve with obstruction by road barriers depends on the overlapping vertical alignment. In addition, the available SSD increases as the overlapping sag curve becomes sharper or as the overlapping crest vertical curve becomes flatter. For the cases of relatively flat horizontal curves overlapping with a sag vertical curve or flat vertical grade and with relatively large barrier offsets, the sight line can cross over the barrier without obstruction. However, in these cases, the sight line should be checked for possible obstruction by objects beyond the barrier. In addition, for left-turn horizontal curves, the sight line should not be allowed to cross over the traveled way of the opposite direction. Example design aids were presented to assist designers with easily checking the available SSD on a horizontal curve corresponding to two specific overlapping vertical alignments (i.e., crest curve with $K = 50$ m and flat grade). In cases in which the available SSD is less than the required one on a certain curve, designers may consider increasing lateral offsets; using different types of barriers, such as the three-strand cable barriers, as long as they provide the same level of protection; or taking necessary measures to reduce drivers' speeds. The charts presented in this paper correspond to road barriers without antiglare screens. Such a feature, which is common on urban freeways and expressways with small medians, can be modeled in SDES.

Similar charts can be developed for a complete set of design aids. Alternatively, the design aids can be developed as computer software. The paper focused only on the available SSD. However, it has been shown that the required SSD would also depend on the vertical alignment (*17*), and the designer should use more accurate values in the design. In addition, the work presented in this paper can be repeated for other types of barriers. SDES can model these barriers, but different elements would need to be created to model each type of barrier. Finally, this paper adopted the deterministic design approach, where each design parameter is assigned one specific value corresponding to a near-critical condition. Combining these near-critical values usually yields a conservative design. Therefore, it has been recommended by many researchers that the deterministic design approach be replaced with a probabilistic, reliability-based approach; see, for example, work by Richl and Sayed (*10*). Similarly, the authors of this paper recommend developing the suggested design aids while considering the variability of the design parameters in a reliability-based design approach. SDES has the capability to adopt the reliability-based design approach by repeating the calculation process at a certain station for a user-specified number of times and using different values of design parameters as elaborated elsewhere (*18, 19*).

ACKNOWLEDGMENT

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Neural Network Modeling of 85th Percentile Speed for Two-Lane Rural Highways

Dharamveer Singh, Musharraf Zaman, and Luther White

The study was undertaken to develop neural network models to predict 85th percentile speed for two-lane rural highways in Oklahoma. Several input parameters, namely, physical characteristics of road, traffic parameters, pavement condition indices, and accident data, were considered in developing the neural network models. The physical characteristics of road include surface width, shoulder type, and shoulder width. The traffic parameters cover average daily traffic (ADT) and posted speed. The pavement condition parameters include skid number and international roughness index (IRI). The location and statewide collision rates (overall, fatal, and injury), and percentage of drivers traveling at an unsafe speed were covered in the accident data. Four models were developed. Model 1 included physical characteristics of road and traffic parameters, including posted speed. Model 2 covered all parameters in Model 1 except posted speed. Similarly, Model 3 considered accident data with all parameters in Model 1. Model 4 used all the parameters in Model 3 except posted speed. Models 1 and 3 were more accurate than Models 2 and 4. The parametric study of the model parameters showed that 85th percentile speed decreased with an increase in the accident rate, ADT, skid number, and IRI. Similarly, widening of the surface and shoulder resulted in a higher 85th percentile speed. It is expected that developed neural network models would be an effective tool for the Oklahoma Department of Transportation and other departments of transportation to enhance traffic safety.

Traffic operation on two-lane rural highways and setting posted speed limits are some of the difficult tasks faced by the Oklahoma Department of Transportation (DOT) and other state DOTs. Although many states, including Oklahoma, use 85th percentile speed as a major factor in determining posted speed for rural highways, other factors, such as pavement width, shoulder type, shoulder width, topography, weather, roadside development, and accident experience, also play an important role in determining the posted speed limits. Because these factors can vary substantially from one state to another, the U.S. Congress approved the National Highway System Designation Act, which allows states to set their own posted speed limits (*1*).

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Some states and local transportation agencies set speed limits for streets and highways on the basis of results of engineering and traffic investigations (*2–4*). For example, a spot speed study is conducted to estimate the distribution of vehicle speeds in a stream of traffic at a particular highway location. Although estimation of speed on the basis of a spot study is highly desirable, conducting such studies at multiple locations may be expensive and time-consuming. State and local transportation agencies do not have the resources to conduct a large number of engineering studies required to respond to all of the speed requests in a timely fashion. Moreover, after collecting the field speed data needed to determine 85th percentile speed on a given roadway, an engineer must then rely on professional judgment to determine whether any other factors (e.g., accident experience, roadway geometry, and adjacent development) warrant the establishment of speed limits lower than the observed 85th percentile speed. Therefore, characterizing the relationship between 85th percentile speed and the roadway characteristics is important in selecting the posted speed limits on highway sections where field surveying is difficult because of resource limitations (*3*).

Transportation Research Circular E-C151 (*5*) provides detailed literature reviews on the existing speed models. Researchers have developed both regression and neural network models for predicting safe speed (*3, 4, 6–14*). Most studies emphasize estimating the speed for horizontal curves considering such factors as degree of curvature, length of curve, and deflection angle (*3, 4, 6–14*). In addition, most existing models are applicable only to predict the speed for cars (*5*). Transportation Research Circular E-C151 (*5*) listed certain limitations of regression models. Therefore, it is recommended that advance modeling tools such as neural network models be used to overcome the limitations of linear regression modeling (*2, 5*). Najjar et al. (*3*) and Zaman et al. (*4*) developed neural network models to predict 85th percentile speed for two-lane rural highways in Kansas and Oklahoma, respectively. It was reported that neural network models can be an effective tool for estimating 85th percentile speed. Similarly, McFadden et al. (*2*) developed neural network-based models and compared them with the regression-based models developed by Krammes et al. (*13*). It was reported that the neural network models performed better than the regression models. In a recent study, Taylor et al. (*6*) developed a speed profile model for construction work zones on highways using the neural network approach. However, these studies did not include any accident data. The authors believe that models based on accident data would be helpful in adjusting or verifying the posted speeds on two-lane rural highways, especially at high crash locations.

The study was undertaken to develop neural network models to predict 85th percentile speed for two-lane rural highways in

Oklahoma with and without considering posted speed and accident data. Four models were developed. Model 1 included physical characteristics of road and traffic parameters, including posted speed. Model 2 covered all the parameters included in Model 1 except posted speed. Similarly, Model 3 considered accident data along with all the parameters in Model 1. Model 4 used all the parameters in Model 3 except posted speed. A parametric study of the model parameters was also conducted. It is expected that the developed model may be useful for future prediction of 85th percentile speed when roadway characteristics or traffic operational factors change.

SELECTION OF MODEL PARAMETERS

The selection of model parameters is a critical task in any modeling process. A literature survey was conducted to examine and identify the roadway and traffic-related variables that affect the setting of speed limits. The following four groups of factors are believed to affect the traffic speed: geometric features, traffic characteristics, environmental conditions, and driver's experience and knowledge of the roadway. A detailed discussion of the factors affecting traffic speed can be found in work by Singh et al. (*15*).

From the literature review, the model parameters were divided into the following four groups: physical roadway characteristics, traffic parameters, pavement condition indices, and accident data. The physical roadway characteristics include surface width, shoulder type, and shoulder width. The traffic parameters category includes average daily traffic (ADT), posted speed, and 85th percentile speed. The pavement condition indices group includes skid number and international roughness index (IRI). The location collision rate, overall; location collision rate, fatal; location collision rate, injury; statewide collision rate, overall; statewide collision rate, fatal; statewide collision rate, injury; and percentage of drivers traveling at an unsafe speed are covered in the accident data. Statewide rates are computed on the basis of similar roadways pertaining to number of lanes, divided or undivided, rural or urban, and access control. The parameter of drivers traveling at an unsafe speed reflects the percentage of crashes that the reporting officer attributed to speed. The parameters for each group are listed in Table 1.

DATA COLLECTION

The University of Oklahoma research team worked closely with the staff of the Oklahoma DOT Traffic Engineering Division to prepare the database used for the development of the neural network models. The site-specific data were collected from different sources: Highway Performance Monitoring System (HPMS) reports (*16*), need and sufficiency reports (*17*), skid number reports (*18*), and speed study reports (*19*). The HPMS reports provide data that reflect the extent, condition, performance, and operating characteristics of the nation's highways (*16*). Need and sufficiency reports provide the sufficiency rating, which is calculated to determine the physical and operational adequacy of a roadway or bridge (*17*). Skid number reports provide the skid number for various control sections of each county (*18*). Accident reports contain various collision rates (overall, fatal, and injury) and drivers traveling at an unsafe speed. The speed study report provides 85th percentile speed and posted speed at a particular location on a selected site (*19*).

Because the purpose of the study was to develop neural network models for estimating 85th percentile speed for two-lane rural highways, only sites for which 85th percentile speeds were available were chosen. Once the site was selected, the other model parameters, namely physical characteristics of road, traffic, road condition, and accident data, were collected from the HPMS report, need and sufficiency report, skid report, and accident report, respectively. Data were collected from 241 sites that span 46 counties and eight divisions in Oklahoma. The study is limited to estimating the speed on tangent sections only. Other parameters, such as driveways, horizontal and vertical curves, percent passing, and passing sight distance, are not included in this study.

Physical Characteristics of Road

The surface width, shoulder width, and shoulder type were recorded from the need and sufficiency report (*17*). The shoulders were categorized in six groups, with each group given a number: no shoulder, 0; paved, 1; gravel, 2; sod, 3; curb on both sides, 4; curb on one side, 5; and combination paved and sod, 6.

TABLE 1 Descriptive Statistics of Model Parameters

Parameter	Designation	Max.	Min.	Mean	SD
Physical characteristics of road	Surface width (SW) (ft)	24	20	23.85	0.68
	Shoulder type (ST)	6	1	na	na
	Shoulder width (SHW) (ft)	10	1	6.12	2.36
Traffic parameters	Average daily traffic (ADT)	9,100	330	3,180	2,005
	Posted speed (PS) (mph)	65	35	55.2	9.2
	85th percentile speed (V_{85}) (mph)	70.5	38.5	57.1	8.8
Pavement conditions	Skid number (SN)	62.8	25.6	42.8	8.2
	International roughness index (IRI) (in./mi)	202	38	96.5	33.0
Accident data	Location collision rate overall[a] (LCRO)	762.4	0	125.7	111.1
	Location collision rate fatal[a] (LCRF)	54.6	0	3.0	6.4
	Location collision rate injury[a] (LCRI)	330.4	0	55.4	46.8
	Statewide collision rate overall[a] (SCRO)	189.1	86.3	116.1	46.7
	Statewide collision rate fatal[a] (SCRF)	3.2	1.9	2.8	0.57
	Statewide collision rate injury[a] (SCRI)	65.2	41	48.5	10.8
	Unsafe speed drivers (USD) (%)	61.9	0	14.9	13.2

NOTE: Max. = maximum; min. = minimum; SD = standard deviation; na = not applicable.
[a]Crashes per 100 million vehicle miles.

Traffic Parameters

The 85th percentile speed and posted speed were recorded from the speed study reports (*19*). Speeds were collected only in the daytime and under favorable weather conditions. Cars and trucks were measured under free-flow conditions. ADT for each site was obtained from the need and sufficiency report (*17*).

Pavement Condition Parameters

The IRI and skid number values were collected from the HPMS report (*16*) and the skid number report (*18*), respectively.

Accident Data

The Oklahoma DOT provided access to its online database for accident data. The following accident-related data were collected: location collision rate, overall; location collision rate, fatal; location collision rate, injury; statewide collision rate, overall; statewide collision rate, fatal; statewide collision rate, injury; and drivers traveling at an unsafe speed.

DESCRIPTIVE STATISTICS OF MODEL PARAMETERS

Descriptive statistics of all model parameters are given in Table 1. The maximum and minimum values of surface width were 20 and 24 ft, respectively. Approximately 98% of the sites had a surface width of 24 ft. The shoulder width ranged from 1 to 10 ft, with an average value of approximately 6 ft. The shoulder types included in the data set are Type 1 (paved), Type 2 (gravel), Type 3 (sod), and Type 6 (combination paved and sod). The shoulder type data indicate that most of the sites had Type 1 shoulder (paved), followed by Type 3 (sod), and Type 6 (combination paved and sod). The maximum and minimum values of skid number and IRI were observed to be 62.8 and 25.6 and 202 and 38 in./mi, respectively. The ADT values ranged between 330 and 9,100, with an average of 3,180. The maximum and minimum posted speed and 85th percentile speed values were found to be 65 mph and 35 mph and 70.5 mph and 38.5 mph, respectively. The location collision rate, overall, values ranged from 0 to 762.4 (per 100 million vehicle miles), and the statewide collision rate, overall, values were in the range of 86.29 to 189.05 (per 100 million vehicle miles). There were little variations observed in the statewide collision rate (statewide collision rate, overall; statewide collision rate, fatal; and statewide collision rate, injury). The percentage of drivers traveling at an unsafe speed was observed in the range of 0 to 61.9.

NEURAL NETWORK MODELING

Description of Models

Four neural network models were developed with and without considering posted speed and accident data. The purpose of including accident data in the modeling was to assess whether inclusion of the accident data affects the performance of the developed models. In addition, it was found that posted speed was highly correlated with 85th percentile speed with a correlation coefficient (R^2) = 0.93.

TABLE 2 Parameters for Neural Network Models

Model Parameter	Model 1[a]	Model 2[b]	Model 3[c]	Model 4[d]
SW	x	x	x	x
ST	x	x	x	x
SHW	x	x	x	x
ADT	x	x	x	x
SN	x	x	x	x
IRI	x	x	x	x
PS	x	—	x	
LCRO	—	—	x	x
LCRF	—	—	x	x
LCRI	—	—	x	x
SCRO	—	—	x	x
SCRF	—	—	x	x
SCRI	—	—	x	x
USD	—	—	x	x

NOTE: — = parameters not considered for modeling.
[a]Model 1: with posted speed but without accident data.
[b]Model 2: without posted speed and without accident data.
[c]Model 3: with posted speed and with accident data.
[d]Model 4: without posted speed but with accident data.

Hence, it was decided to separate the models on the basis of the posted speed. The developed neural network models provide the flexibility of using selected input parameters in estimation of 85th percentile speed. Table 2 lists the independent parameters used for developing each model. The cross mark (x) indicates that the model parameters are included in the modeling process as input variables.

Division of Data Set

The first step in the formulation of a neural network model is to separate the data set into two subsets, one for training and one for testing of the developed model. This separation should be done randomly, but it should also be done in a manner such that the training data set has the range of variables seen in the testing data set or expected to be seen in further applications of the model (*4, 20*). Consequently, the overall data set of 241 sites was divided into training and testing data sets. Roughly 80% (193 sites) and 20% (48 sites) of the total data were used for training and testing purposes, respectively. Each variable was normalized so that the mean and variance became zero and unity, respectively. Normalization of the input and output variables is an important step to achieve the expected performance of the neural network models (*20*).

Neural Network Model Formulation

Neural network models represent a class of model designed to perform mapping of an input vector into an output vector (*4, 20*). The architecture of a simple neural network model is a collection of nodes distributed over an input layer, hidden layer or layers, and an output layer (Figure 1). In this study, a three-layer (one input, one hidden, and one output) network was used (Figure 1). The input layer contains the independent variables (Model 1: 7, Model 2: 6,

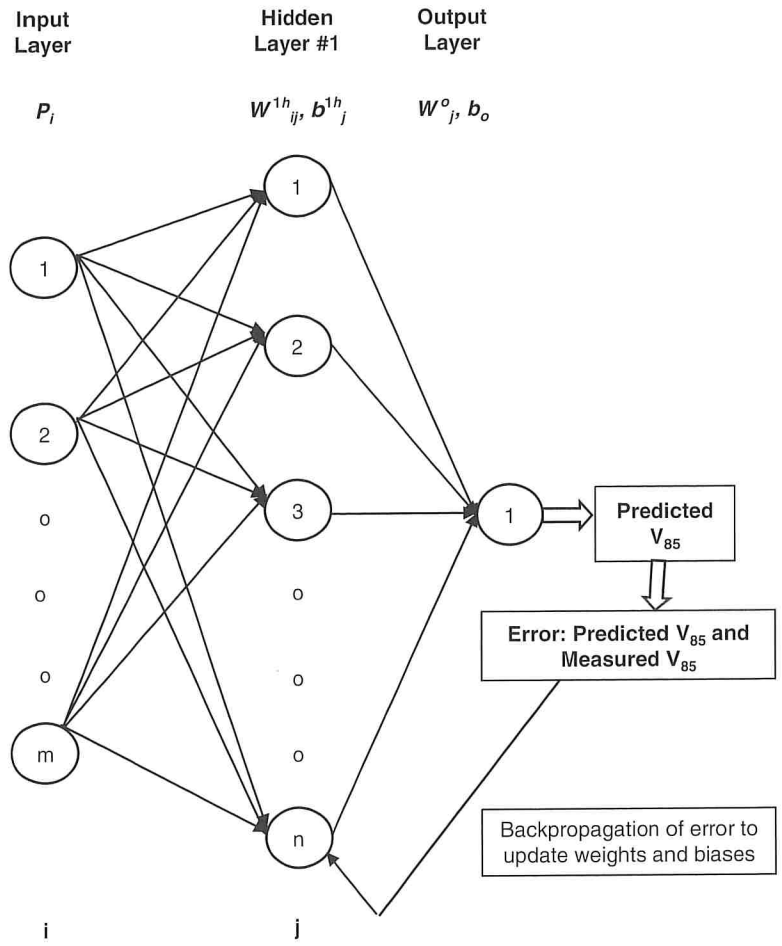


FIGURE 1 Neural network flow diagram.

Model 3: 14, and Model 4: 13 input variables), and the output layer contains the dependent variables (i.e., 85th percentile speed). The output for this configuration of the network was calculated using Equation 1 and is a function of the input variables. A hyperbolic tangent (called tansig) function was used as an activation function for the hidden layer (Equation 2), and a pure linear function (called purelin) was used for the output layer (Equation 3).

$$O(P_1,\ldots,P_m) = f_o\left[b_o + \sum_{j=1}^{n}\left\{W_j^o f_h\left(b_j^{1h} + \sum_{i=1}^{m} W_{ij}^{1h} P_i\right)\right\}\right] \quad (1)$$

$$f_h(T) = \frac{2}{1+e^{-2T}} - 1 \quad (2)$$

$$f_o(T) = T \quad (3)$$

where

i = subscript for input layer,
j = subscript for first hidden layer,
m = number of input parameters,
n = number of nodes in first hidden layer,
f_h = transfer function for hidden layers,
f_o = transfer function for output layer,
W_{ij}^{1h} = weight factors for first hidden layer,
W_j^o = weight factors for output layer,
b_j^{1h} = bias factors for first hidden layer,
b_o = bias factor for output layer,
P_i = input variables (Table 2), and
O = output value (i.e., 85th percentile speed).

The next step was to select a training algorithm. In the context of neural networks, supervised learning algorithms actually are algorithms that iteratively select network weights to reduce the squared error between network outputs and observations. Generally, in any learning algorithm, the input vectors are presented to the network to obtain an associated output. The difference between the target output and the network output represents the error. The squared error is minimized by adjusting network weights and biases with an iterative optimization algorithm. Iterations continue until the error goal is reached. The back-propagation method is one such method that has been traditionally used. However, because of its efficiency, the Levenberg–Marquardt optimization algorithm was used for the training of the network. The neural network codes were developed using commercial software called MATLAB. The trained models were tested for 48 sites that were not used in the training stage. The strength of each training and testing stage was evaluated by calculating the percentage mean absolute relative error (MARE). The lower value of MARE

represents that the model would result in lesser amount of error in the prediction.

Methodology to Estimate Network Output

Generally, the output of the neural network depends on the initialization of the weights in the minimization algorithms (*20*). Ideally, the network should give close to the same answer regardless of the initial value of the weight, but in reality that may not be the case. This result could be because the error function may have multiple minima; it is also possible that the optimization method is local in nature. Therefore, it is important to conduct several trials with different initial weights to get the global minima. In this paper, a statistical approach was used to get the final output from the network. The approach used involved random generation of starting values of weights in the iterative training algorithms to obtain a collection of different estimated weights. This collection is used in simulation to determine distributions of outputs for a given input. Each model was developed by randomly varying the weight 500 times, and then the output was calculated as the mean of all 500 values from the histogram. This approach gives a measure of the uncertainty in the predicted output for the new data.

Design of Network Architecture

The accuracy of the neural network model depends on the network architecture. The network architecture indicates the number of hidden layers and number of neurons in each hidden layer. There is no specific rule to select the number of hidden layers and neurons. After numerous trial and error processes, the best performance was found with one hidden layer having six neurons. The number of nodes in the input layer depends on the number of model parameters (i.e., Model 1: 7, Model 2: 6, Model 3: 14, and Model 4: 13 input variables). The number of nodes in the output layer was one (i.e., 85th percentile speed). The architecture of a particular model can be presented in the form of I-H-O, where I, H, and O indicate the number of nodes in input, hidden, and output layers, respectively. For example, Model 1 has 7-6-1 architecture, where 7 indicates the input parameters, 6 indicates one hidden layer with six neurons, and 1 indicates the output layer with one neuron (i.e., 85th percentile speed). Similarly, Model 2, Model 3, and Model 4 can be represented as 6-6-1, 14-6-1, and 13-6-1, respectively.

DISCUSSION OF RESULTS

Model 1. With Posted Speed but Without Accident Data

The performance indicator, MARE, for the training data set was found to be 2.6% (i.e., ≈ 97% degree of accuracy), indicating that this model is well trained to calculate 85th percentile speed. Figure 2*a* shows the histogram of the training state. Model 1 predicted 85th percentile speed with an absolute speed difference (|measured 85th percentile speed – predicted 85th percentile speed|) of less than 5 mph for 98% of the sites (Figure 2*a*). Figure 2*b* shows the histogram of the testing data set for all 48 sites. Model 1 predicted 85th percentile speed with an absolute speed difference of less than 5 mph for almost 90% of the sites with a MARE value of 5.1%. The results

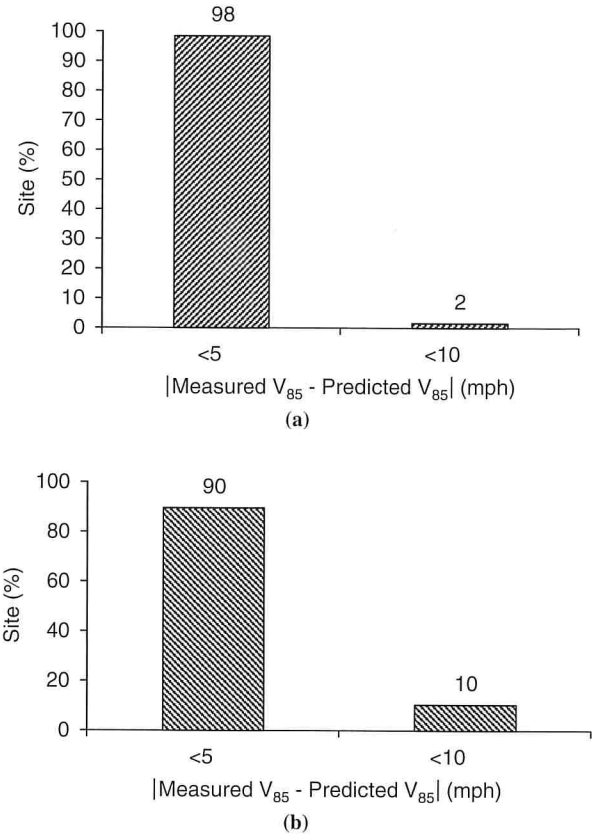


FIGURE 2 Histograms for Model 1: (*a*) training data set, MARE = 2.6%, and (*b*) testing data set, MARE = 5.1%.

indicate that this model is excellent in predicting 85th percentile speed. This model can be used with a minimum of available inputs (Table 2).

Model 2. Without Posted Speed and Without Accident Data

Figure 3*a* shows the histogram for the training data set. Model 2 predicted 85th percentile speed with an absolute speed difference of less than 5 mph and within a range of 5 to 10 mph for 68% and 25% of the sites, respectively. The MARE value for the training data set of this model was 7.4% (i.e., ≈ 93% degree of accuracy). The performance of this model for the testing data set was reasonably good (but worse than Model 1), with an absolute speed difference of less than 5 mph for almost 50% of the sites and a MARE of 13.8% (Figure 3*b*). Model 1 has a higher accuracy rate than Model 2. It is expected that the inclusion of posted speed in Model 1 produces a slightly higher rate of accuracy. Model 1 and Model 2 can be used to establish the range of 85th percentile speed with and without using posted speed.

Model 3. With Posted Speed and with Accident Data

Figure 4*a* shows the histogram for the training data set, which indicates that this model is expected to predict 85th percentile speed

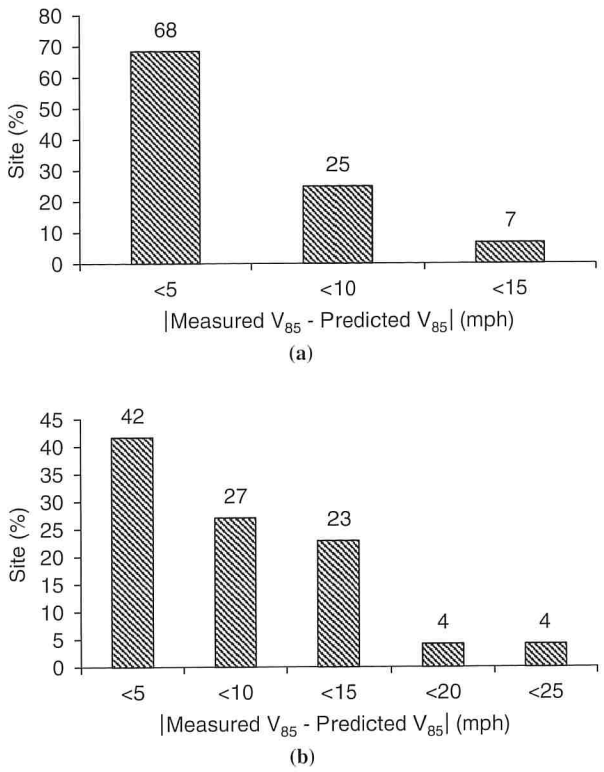


FIGURE 3 Histograms for Model 2: (*a*) training data set, MARE = 7.4%, and (*b*) testing data set, MARE = 13.8%.

with an absolute speed difference of less than 5 mph for 99% of the sites. The MARE value for the training data set was 1.8%, which indicates that this model is well trained to estimate 85th percentile speed with a degree of accuracy of approximately ±98%. Figure 4*b* shows the histogram of the testing data set. It shows that Model 3 predicted 85th percentile speed with an absolute speed difference of less than 5 mph for almost 83% of the sites with a MARE of 5.8%. The inclusion of accident data slightly improves the performance of Model 3 as compared with Model 1 in the training stage. However, in the testing stage, Model 3 performs poorly compared with Model 1. Model 3 is useful when accident data are available for a selected site, and Model 1 or Model 2 may be used in absence of accident data.

Model 4. Without Posted Speed but With Accident Data

Figure 5*a* shows the histogram for the training data set, which indicates that Model 4 is expected to predict 85th percentile speed with an absolute speed difference of less than 5 mph for approximately 90% of the sites. The MARE value for the training data set was 4.4% (i.e., ≈ 95.6% degree of accuracy). Figure 5*b* shows the histogram of the testing data set. This model predicted 85th percentile speed with an absolute speed difference less than 5 mph for almost 56% of the sites with a MARE of 12.3%. A comparison of Model 2 and Model 4 shows that for the training data set, the percentage sites having absolute speed difference of less than 5 mph increased from 68% to 90%, which seems a significant improvement over Model 2.

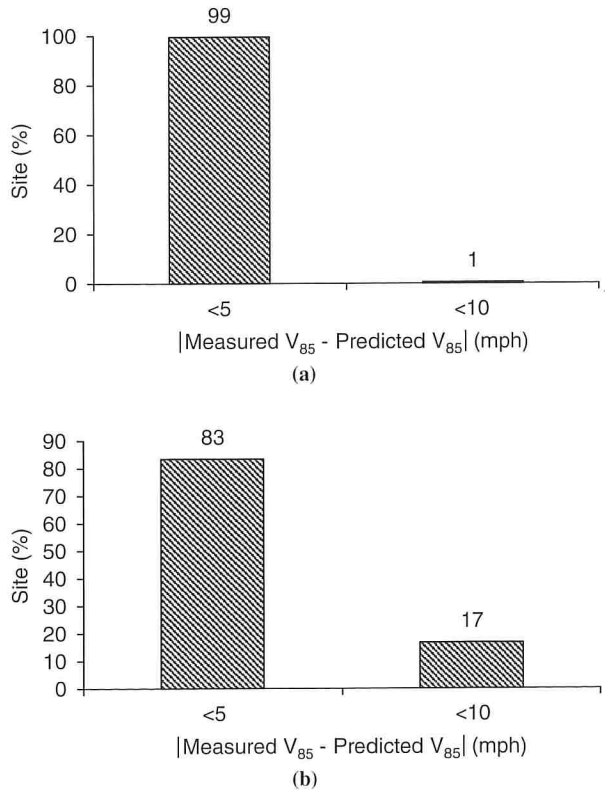


FIGURE 4 Histograms for Model 3: (*a*) training data set, MARE = 1.8%, and (*b*) testing data set, MARE = 5.8%.

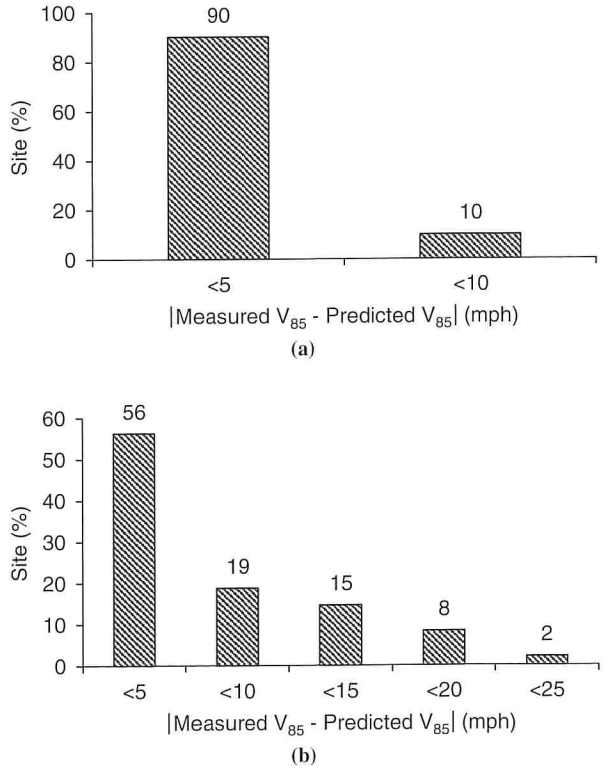


FIGURE 5 Histograms for Model 4: (*a*) training data set, MARE = 4.4%, and (*b*) testing data set, MARE = 12.3%.

Similarly the performance of Model 4 for the testing data set shows that inclusion of accident data marginally improves the performance of this model when compared with Model 2. Model 4 is useful when the accident data are available for a site. In the absence of accident data, Model 2 can be used with a reasonable level of accuracy.

PARAMETRIC STUDY

A parametric study was conducted to obtain an understanding of the relationship between 85th percentile speed and the other model parameters. Posted speed was highly correlated with 85th percentile speed with a correlation coefficient (R^2) = 0.93; hence, researchers decided to use a model that does not include posted speed for the parametric study. Therefore, Model 3 was used to demonstrate the effect of different parameters on 85th percentile speed, which does not consider posted speed. The selected model parameter was varied, and the other parameters were kept to their average value. Shoulder Type 1 (paved shoulder) was kept constant for all cases.

Physical Characteristics of Road

Figure 6*a* shows the variation of 85th percentile speed with surface width. A clear trend is not evident because of the limited range of surface width (i.e., 20 to 24 ft) in the study (Table 1). It is expected that on a wider road a driver feels more comfortable and does not perceive any danger, which results in a higher speed. Inclusion of additional sites may be required to demonstrate the effect of surface width on 85th percentile speed.

Figure 6*b* shows variation of 85th percentile speed with shoulder width. A trend line was fit to the data points. Figure 6*b* shows that an increase in shoulder width results in a higher 85th percentile speed; 85th percentile speed is constant for a shoulder width less than 4 ft, and then it increases with an increase in shoulder width. It is believed that an increase in shoulder width changes drivers' perception and encourages them to drive faster than they do when the shoulder is narrow (*3, 21*).

Variation of 85th percentile speed with shoulder type is shown in Figure 6*c*. Four different types of shoulders are selected: Type 1 (paved), Type 2 (gravel), Type 3 (sod), and Type 6 (combination of paved and sod). Two-lane rural roads with Type 1 (paved) shoulders result in a higher 85th percentile speed compared with the other types of shoulders (Type 2, Type 3, and Type 6). Type 2 (gravel) and Type 3 (sod) shoulders exhibit speeds that are lower by about 7 mph and 13 mph, respectively, than Type 1 (paved) shoulders. Although not expected, Type 6 shoulders (combination of paved and sod) results in the lowest 85th percentile speed. It is expected that drivers would consider paved shoulder the safest, and hence the driving speed would be higher (*21*). With gravel, sod, and combination shoulders, drivers are likely to become more cautious, which would cause a reduction in 85th percentile speed.

Traffic Parameters

Figure 7 shows that 85th percentile speed decreases with an increase in ADT. For example, increasing ADT from 2,000 to 5,000 results in a reduction in 85th percentile speed from 71 mph to 51 mph. It is expected that a higher traffic volume would significantly increase the queue length and the increased queue length would cause a driver to drive slowly, consequently resulting in a slower 85th percentile speed (*3*). In addition, drivers may perceive that the speed limit is more strictly enforced on roadways with higher ADT and may drive slowly to avoid being ticketed for speeding (*7*).

Pavement Conditions

Drivers' speed largely depends on pavement conditions. For example, a smoother pavement would tempt drivers to drive faster, whereas a rough pavement (higher skid number or IRI) would force drivers to drive slowly. IRI is also used by FHWA to assess changes in the condition of the nation's highways and to forecast highway investment needs.

Variation of 85th percentile speed with two different pavement condition indices, skid number and IRI, are given in Figure 8, *a* and *b*, respectively. As expected, 85th percentile speed decreases with an increase in road roughness. For example, an increase in skid number from 30 to 60 decreased 85th percentile speed from 60 mph to 40 mph. Similarly, for smoother pavements with IRI < 60 in./mi, 85th percentile speed was above 68 mph; however, it dropped to 50 mph when the IRI value went above 200 in./mi.

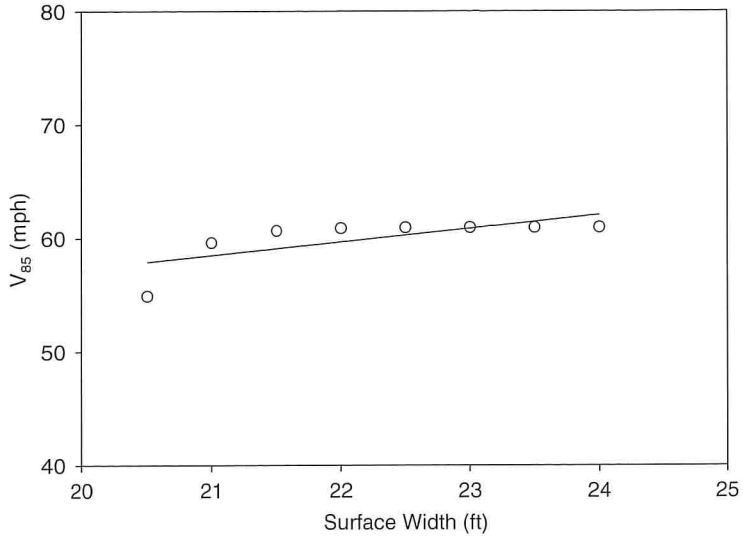
Accident Parameters

Figure 9, *a* and *b*, shows the variation of 85th percentile speed with location collision rate, overall, and statewide collision rate, overall, respectively. An increase in location collision rate, overall, and statewide collision rate, overall, causes a reduction in 85th percentile speed. For example, with zero location collision rate, overall (no accident), 85th percentile speed was calculated to be approximately 64 mph, and it decreased to 41 mph when location collision rate, overall, increased to 700 (100 million vehicle miles) (Figure 9). Similar observation can be made for statewide collision rate, overall (Figure 9*b*). The accident models may be helpful in adjusting or verifying the speed at high crash locations.

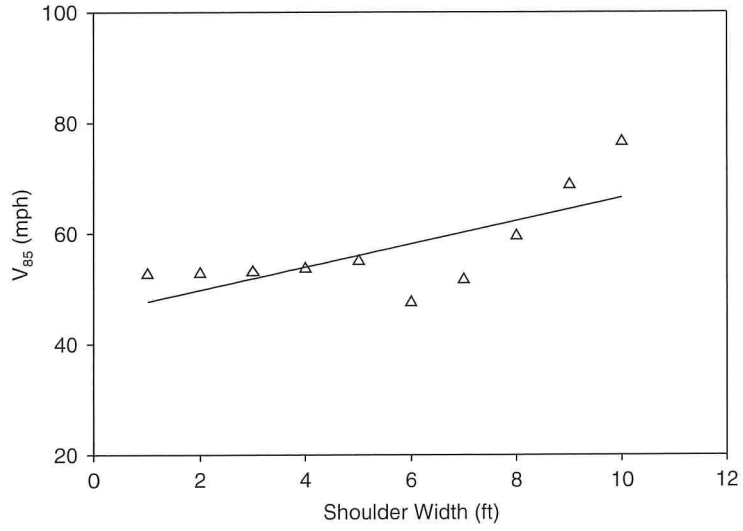
CONCLUDING REMARKS

The developed neural network models are expected to be a useful and robust tool to help Oklahoma DOT traffic engineers in performing any of the following tasks:

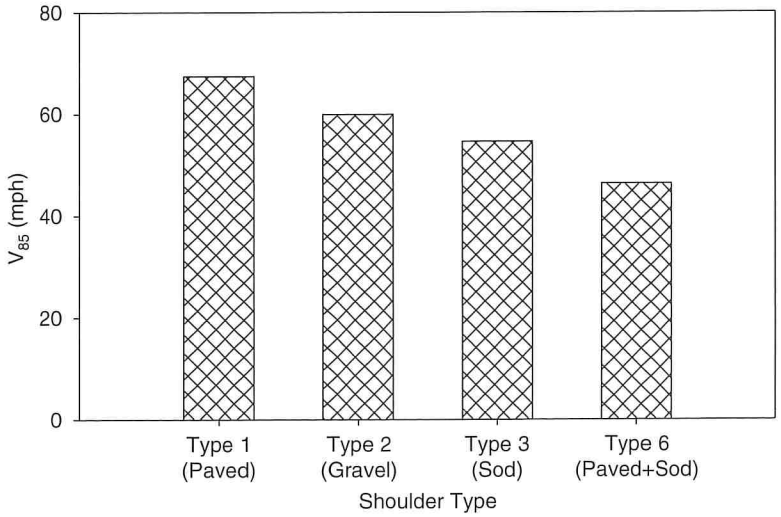
- Predict 85th percentile speed, with reasonable accuracy, on the basis of a few roadway and traffic parameters without performing a costly 85th percentile speed measurement (spot study), especially in situations where time and money are scarce.
- Validate field observations and studies and, therefore, assess any deviation from the neural network model prediction. Investigate the impact of any proposed changes or variations in roadway-related input parameters on 85th percentile speed at a given section of a two-lane state highway.
- The developed neural network models provide flexibility of using selected input parameters in estimation of 85th percentile speed. Model 1 and Model 2 should be used in the absence of accident data. Model 3 and Model 4 should be used when accident data are available.
- The parametric study of the model parameters showed that 85th percentile speed decreased with an increase in accident rate,



(a)



(b)



(c)

FIGURE 6 Variation of 85th percentile speed with (*a*) surface width, (*b*) shoulder width, and (*c*) shoulder type.

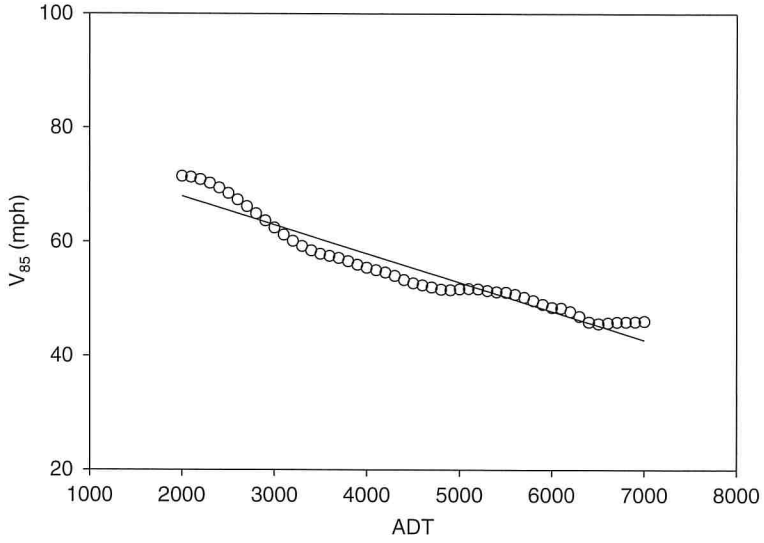
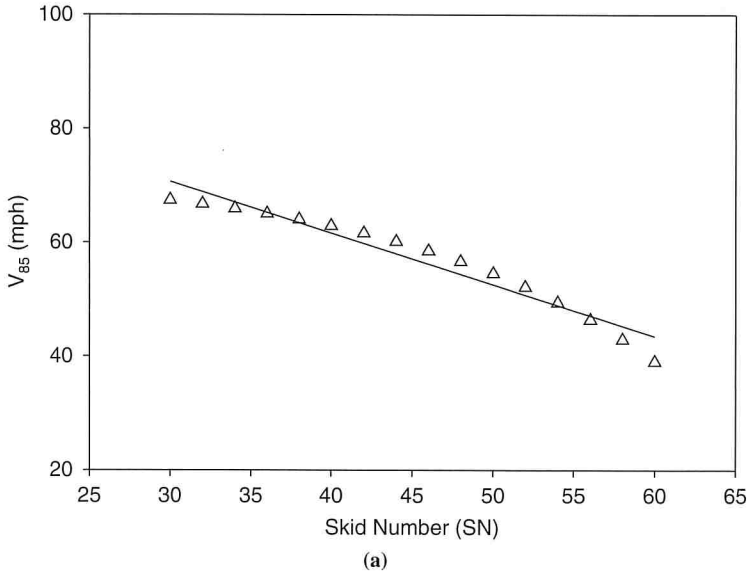
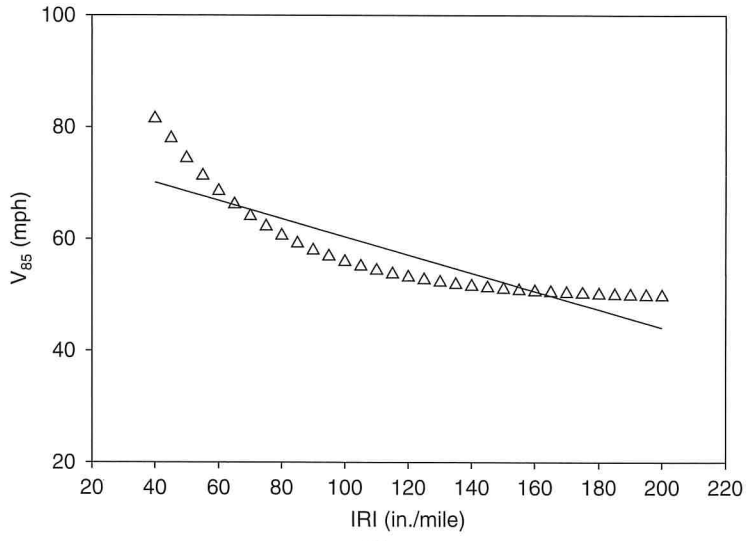


FIGURE 7 Variation of 85th percentile speed with ADT.



(a)



(b)

FIGURE 8 Variation of 85th percentile speed with (*a*) skid number and (*b*) IRI.

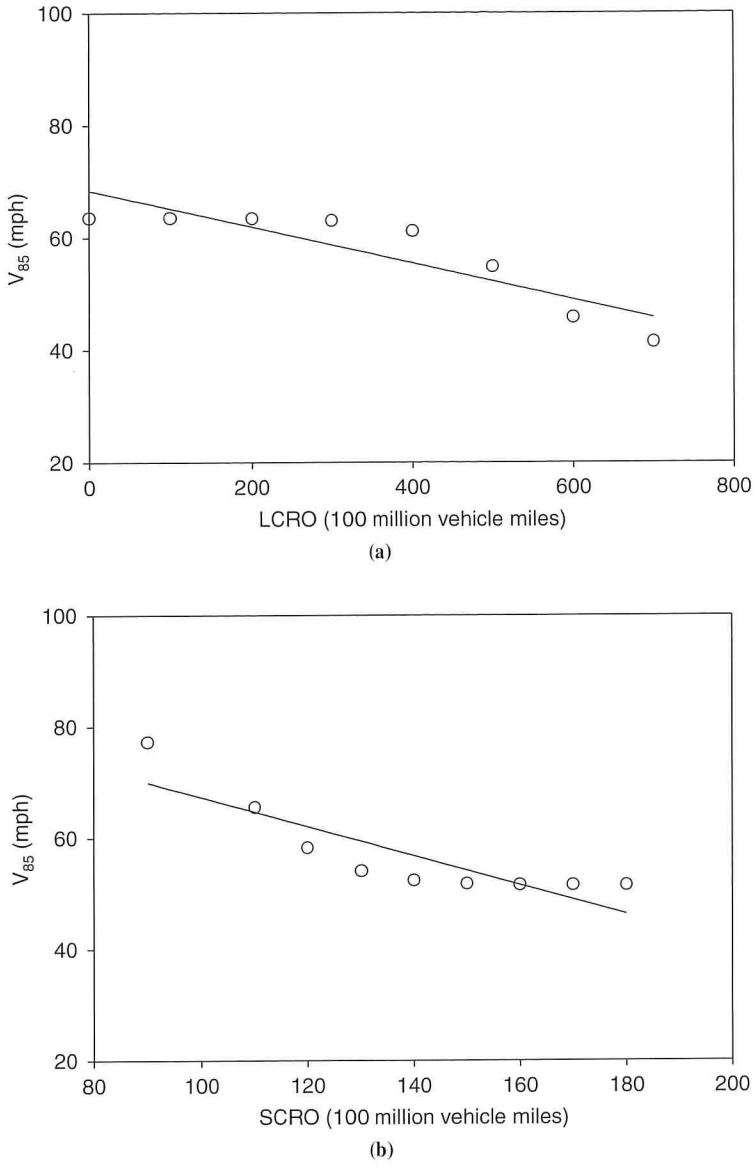


FIGURE 9 Variation of 85th percentile speed with (*a*) location collision rate, overall, and (*b*) statewide collision rate, overall.

ADT, skid number, and IRI. Similarly, an increase in surface width and shoulder width resulted in a higher 85th percentile speed. The type of shoulder affected 85th percentile speed significantly.

The main focus of the study was to develop neural network models for the estimation of 85th percentile speed for two-lane rural highways. The study was limited to estimating the speed only on tangent sections. Other parameters such as driveways, horizontal and vertical curves, percentage passing, and passing sight distance may be included in future studies.

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The Operational Effects of Geometrics Committee peer-reviewed this paper.

Road Cross-Section Width and Free-Flow Speed on Two-Lane Rural Highways

Pedro Melo, António Lobo, António Couto, and Carlos Manuel Rodrigues

Speed choice is strongly influenced by geometric road features. In this work, the influence of lane and shoulder widths on free-flow speed was studied with the driving simulator DriS at the University of Porto in Porto, Portugal. To evaluate how speed was influenced by the cross section, this study investigated the possible influence of the order of magnitude of the practiced speeds on the effects of variations in lane and shoulder widths. Two types of roads with different base speeds were considered. The roads were presented to drivers on a driving simulator. The validity of the data obtained in the simulator was confirmed through a comparative analysis of the registered speeds in the real environments for the equivalent simulator conditions at six points of control. The lane and shoulder widths from which the free-flow speed was no longer conditioned by the dimensions of the road's cross section were obtained, as well as the reduction in speed associated with smaller widths. Contrary to what was suggested by the *Highway Capacity Manual 2010,* the individual effects of variations in lane and shoulder widths were not cumulative; greater impacts on free-flow speed were obtained by their simultaneous variation.

An important component in characterizing the service provided by a given highway is the free-flow speed (*1*), which represents drivers' speed choice under free-flow conditions. The free-flow speed reflects drivers' response to road geometric and environmental characteristics without the presence of vehicles ahead. The free-flow speed can be also affected by vehicle characteristics and general driving practices. According to the *Highway Capacity Manual 2010* (HCM) (*1*), the level of service for two-lane highways depends on the average travel speed, which in turn depends on the free-flow speed. When it is not possible to conduct the measurements required for calculating the free-flow speed, the HCM (*1*) proposes an estimation model in which some correction values representing road geometric characteristics are applied to a base free-flow speed. This model is represented by Equation 1:

$$FFS = BFFS - f_{LS} - f_A \quad (1)$$

where

FFS = free-flow speed,
BFFS = base free-flow speed,
f_{LS} = adjustment parameter for lane and shoulder width (in kilometers per hour), and
f_A = adjustment parameter due to density of access points (in kilometers per hour).

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The base free-flow speed can be assumed as the speed observed for roads presenting the basic requirements of the geometric conditions suggested by the HCM (*1*): no access points and lane and shoulder widths equal to or greater than 3.6 m and 1.8 m, respectively. For smaller cross sections and higher densities of access points, the HCM (*1*) proposes reductions in the free-flow speed. The reductions related to the width of the cross section are presented in Table 1.

Speed reductions presented in the HCM (*1*) are based on the findings of Harwood et al., presented in the final report of NCHRP Project 3-55(3) (*2*). In this report, a regression relationship between shoulder width reduction and speed reduction observed in a real environment was performed; it assessed the effects of lane width variation on free-flow speed on the basis of previous studies in the area. The values in Table 1 suggest cumulative effects on the free-flow speed from variations in lane and shoulder width (i.e., the reduction in speed for a given cross section composed of a lane width smaller than 3.6 m and a shoulder width smaller than 1.8 m is the sum of the individual effects caused by each variable).

This paper contributes to the evaluation of the effects on the free-flow speed from road cross-section characteristics—lane and shoulder width—through a driving simulation study. The simulated environment allows the assessment of speed reductions for a greater number of cross-section combinations as well as the consideration of a wider range of lane width values than the methodology used in NCHRP Project 3-55(3) (*2*). In addition, the smallest cross section from which the speed choice is no longer affected by the lane and shoulder widths is provided and compared with the HCM (*1*) proposals.

The establishment of free-flow conditions is defined by the time interval between two successive vehicles. The HCM (*1*) suggests that a 3-s headway is suitable for free-flow conditions, although this value is not unanimous among the literature. For example, a study of Portuguese two-lane rural roads by Lobo et al. (*3*) suggests a 6-s gap as the reference for free-flow conditions.

This study contributes to the knowledge of adequate cross-section characteristics for the desired speed of a given road. Several studies on road safety have referred to speeding as a major cause of car accidents (*4, 5*). Therefore, the road geometric features (the horizontal and vertical alignments and cross sections) should suggest to drivers an adequate speed choice to promote road safety and the sustainability of the surrounding environment.

DATA COLLECTION

Experimental Approach

Studies of speed can be performed with different methodologies, such as the use of instrumented vehicles, naturalistic studies, real environment monitoring, or driving simulation, depending on the variables to be considered.

TABLE 1 Adjustment Factor for Reductions in Free-Flow Speed (km/h) According to Lane and Shoulder Widths (*1*)

	Shoulder Width (m)			
Lane Width (m)	≥0.0 to <0.6	≥0.6 to <1.2	≥1.2 to <1.8	≥1.8
2.7 to < 3.0	10.3	7.7	5.6	3.5
≥3.0 to <3.3	8.5	5.9	3.8	1.7
≥3.3 to <3.6	7.5	4.9	2.8	0.7
≥3.6	6.8	4.2	2.1	0.0

TABLE 2 Cross-Section Scenarios Presented in Simulated Environment

	Shoulder Width (m)							
	N 105-2				N 222			
Lane Width (m)	0.3	0.8	1.2	1.8	1.2	1.8	2.2	2.5
2.7	9	8	10	12	9	8	10	12
3.0	7 (II; III)	1	2	11	7	1	2	11
3.3	4	3 (I)	13	15	4	3	13	15
3.6	6	5	16	14	6	5	16 (I; II)	14 (III)

NOTE: Observations I, II, and III—cross sections considered for validation with the real environment spot speeds.

This study analyzed the average travel speed adopted for a set of road scenarios with different cross sections. The use of a driving simulator allowed a high level of control over the variables (*6*). Therefore, it is the most suitable approach for studying changes in drivers' speed choice as a result of the variation of the characteristics of the cross section.

The simulator used in this study was the DriS, which is a fixed-base driving simulator installed at the Traffic Analysis Laboratory of the Faculty of Engineering of the University of Porto in Porto, Portugal. Two roads near Porto (N 105-2 and N 222) were selected for this study. The design speeds of these roads are 40 km/h and 80 km/h, respectively, representing two types of roads: one more winding and the other less demanding for drivers. For each road, the road alignment of a 3-km-long section was collected by a Global Positioning System device installed in an instrumented vehicle. Then, both stretches were reproduced in the DriS to compare the influence of variations in the cross-section width on the free-flow speed of roads presenting different alignment standards. It was expected that the effects of the characteristics of the cross section were greater for higher speeds.

The driving experiences conducted in the DriS were performed by 15 drivers. The group of drivers had the following characteristics: (*a*) 60% were males and 40% were females; (*b*) they ranged in age from 20 to 40 years; (*c*) each driver had held a driver's license for 3 or more years; (*d*) each driver drove at least 10,000 km per year, with 5,000 km on two-lane rural highways; and (*e*) they did not usually drive on the chosen roads. The 15 drivers were selected because they adapted well to the simulated environment and showed realistic driving behaviors.

The drivers drove the DriS across the simulated roads, composed of successive 3-km-long sections, each one presenting a different combination of lane and shoulder width. The characteristics of the horizontal alignment were kept the same. Smooth cross-section transitions between scenarios were ensured by intermediate 100-m-long tangent sections. The lane and shoulder widths are presented in Table 2.

The values considered for lane and shoulder widths were chosen according to the HCM (*1*) (Table 1) and according to the cross-section characteristics most commonly adopted on these types of roads. For each road, the 16 scenarios were driven in sequences of four to avoid driver fatigue. These sequences were chosen randomly to avoid excessive familiarity with the road alignment, and they were preceded by 5-km-long training stretches.

The realism of the simulated environment was improved through the introduction of trees at the roadside and of opposing traffic appearing in the same spot of the road for all the scenarios. The presence of vehicles ahead was not introduced in the simulated environment. In these conditions, the average travel speed corresponds to the free-flow speed.

The selected 3-km-long stretches were built on level terrain, with grades below 3% (*1*). Therefore, the effects of the vertical alignment were not considered in the simulator study. Moreover, no intersections, which could disturb drivers' speed choice, were present in the road sections under consideration.

In Figure 1, the simulation setting and screen are presented.

Description of Variables

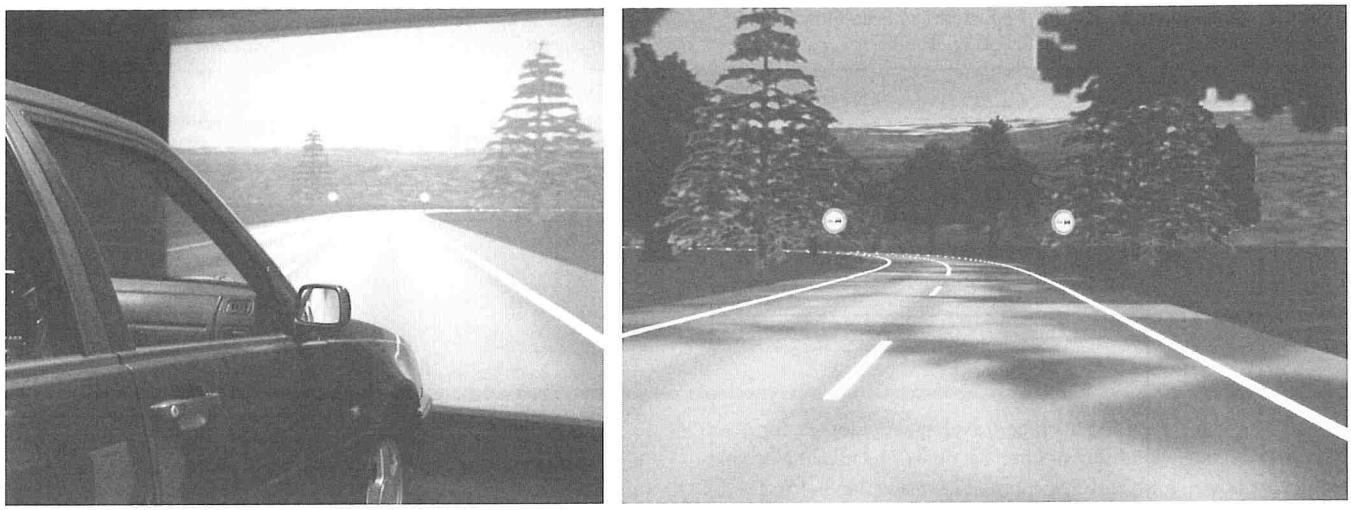
The variables considered in this study were evaluated using the vehicle's spatial and temporal location, collected by the DriS.

First, a set of three variables was considered to evaluate the quality of each experiment, removing drivers who had adopted unrealistic behaviors. Thus, using the registered number of times that drivers crossed the lane limits, two variables were created: the average percentage of road length driven crossing the centerline and the average percentage of the road length driven crossing the sideline. These variables are mutually exclusive, and their sum represents the average percentage of road length driven out of the lane limits. For both roads, the maximum driving error allowed for the drivers was approximately 30%.

The average distance to the centerline and the average distance to the sideline were estimated for a sensitivity analysis of the driver's behavior when faced with different cross-section widths. For this estimation, the distances registered out of lane limits were considered to be negative.

For the development of the regression model, the dependent variable was the free-flow speed obtained in the simulated environment, which is represented by the average travel speed estimated for each cross-section combination, with the length of the stretch divided by the average travel time observed for each driver.

The independent variables were lane width, shoulder width, the product of the lane and shoulder widths, and the dummy variables representing the different roads and drivers. The product of the lane and shoulder widths allowed the evaluation of the combined effects of both variables. The dummy variables for the roads allowed the inclusion of all gathered data into the same regression, increasing the number of observations. Otherwise, some combinations of lane



(a) (b)

FIGURE 1 DriS simulation setting and screen.

and shoulder widths would be repeated without being distinguished by the road type. Dummy variables related to drivers express the individual driving behavior observed for each person when compared with a randomly chosen base driver. Thus, only 14 dummy variables for the drivers were considered.

Finally, the speed values obtained in the simulated environment were calibrated. Therefore, the average spot speed in the real environment was determined for three elements (tangents and curves) of each road. The same speed was then determined in the simulated environment for the same elements (with the same cross-section width), allowing the comparison between both environments.

METHODOLOGY PROPOSED

Regression Modeling

The proposed model assumes that the relationship between the free-flow speed and the independent variables is suitable to be represented by a power function that can be linearized by extracting the natural logarithm of the continuous variables, as presented in Equation 2:

$$\ln(\text{FFS}) = a + b\ln(l) + c\ln(s) + d\ln(l)\ln(s) + ed_r + \sum_{i=1}^{14} f_i d_{di} \quad (2)$$

where

FFS = free-flow speed,
a, b, c, d, e, f_i = regression coefficients,
l = lane width,
s = shoulder width,
d_r = dummy variable for road type, and
d_{di} = dummy variable for drivers.

A multiple linear regression (the least squares approach) was performed with Equation 2 to estimate the effects of the cross section on the free-flow speed.

Then, for each cross-section width and road type, the average free-flow speed value of the 15 drivers was estimated. The minimum cross section from which the increase in lane and shoulder widths did not produce effects on the free-flow speed was considered the baseline scenario. This scenario serves as a reference for estimating the reductions in the free-flow speed for the geometric characteristics of narrower cross sections.

Validation of Results

The road sections tested in the simulated environment were reproductions of real roads, allowing the results obtained in the simulator to be validated (*7*). The validation was performed through a comparative analysis of the spot speeds observed in the real and simulated environments at six points of control (represented in Table 2 by the numerals I, II, and III for each road).

In the simulated environment, spot speeds were automatically registered by the DriS. In the real environment, spot speeds were collected with traffic-counting devices, consisting of a Doppler radar sensor with integrated flash random access memory and a real-time clock. These devices were placed at approximately a half-extension of the road elements used for the validation. The presence of the traffic counters was disguised to avoid biasing driver behavior.

The vehicles used for the free-flow speed estimation in the real environment presented headways equal to or greater than 10 s to ensure free-flow travel conditions.

APPLICATION PERFORMED

Variable Estimation

The experimental study was performed in the DriS, and the variables were obtained for each driver. In Tables 3 and 4, the average variable values for the entire set of drivers are presented for each road and cross section under consideration. As expected, higher speeds were practiced on the road with a less demanding horizontal alignment.

Vehicle Positioning in Cross Section

In Figure 2, the variation in the average distance to the sideline with the cross-section width is presented. The reference values for the distance to sideline, established for a vehicle traveling at the center of the lane, are represented by the dashed line.

For large cross-section widths, drivers tended to travel closer to the sideline because of higher horizontal clearances caused by the increase in shoulder width. As the lane and shoulder width decreased, drivers tended to move toward the centerline to attempt to maintain the same speed, holding the distance to the objects on the roadside. As a consequence of the smaller distance to the opposing traffic stream, drivers tended to reduce speed in the presence of oncoming vehicles.

Multivariate Analysis

To evaluate the effects of the lane and shoulder widths of both road types on the free-flow speed, a multivariate linear regression using Equation 2 was performed with the data collected in the simulated environment. The model was conducted using the statistical software for econometric analysis Limdep (*8*). The regression coefficients and corresponding *P*-values for the set of considered variables are presented in Table 5.

The majority of the considered variables are statistically significant at a 10% level. The exceptions were Driver 1 and four dummy variables. However, the corresponding coefficients were nearly zero, revealing that the behavior of these drivers was similar to that of the base driver.

The speed practiced by the most aggressive driver was around 30% higher than the speed adopted by the least aggressive driver.

The goodness of fit was evaluated with the correlation coefficient ($R = 0.89$). According to Cohen (*9*), a coefficient value above 0.5 represents large correlations between the variables. The coefficient of determination (R^2) expresses that 79% of free-flow speed variance is explained by the variance of the independent variables.

For the road with the lower design speed ($d_r = 0$), the free-flow speed was obtained for each cross section by averaging the free-flow speed practiced by the 15 drivers. The same procedure was adopted for the road with the higher design speed ($d_r = 1$).

The Kolmogorov–Smirnov test was performed for the 32 samples of the 15 speed values obtained in each cross section. Each sample was accepted as being normally distributed for a significance level of 5%, ensuring that the average speed value was representative of the sample.

The elasticities at the sample mean were 0.12% for lane width and 0.025% for shoulder width. These values represent the increase in free-flow speed caused by a 1% increase in lane or shoulder width. However, it is more important to observe the variations in speed produced by different combinations of lane and shoulder widths than to analyze the effects at the sample mean. Thus, Equation 2 was applied to diverse combinations of cross sections. The speed reduction results are presented in Table 6.

Some observations arise from a comparison of Table 1 and Table 6. First, the baseline scenario, corresponding to the minimum cross section, above which significant variations on free-flow speed (<0.5 km/h) are no longer observed, is established in this study for a lane 3.6 m wide combined with a shoulder 0.8 m wide. These baseline widths are valid for both of the studied road types. For wider cross sections, the free-flow speed was strongly influenced by geometric characteristics other than lane and shoulder widths. The base cross section proposed by the HCM (*1*) is formed by a lane 3.6 m wide and a shoulder 1.8 m wide. Because this reference manual was developed for North American conditions, where vehicles are generally wider than in Europe, the minimum cross section not influencing the free-flow speed was expected to be smaller in Portugal.

It is also possible to confirm that narrower cross sections produce greater effects on the free-flow speed. These effects were gradually reduced with increases in lane and shoulder widths. This study obtained values of speed reduction that were different from those of the HCM (*1*). The speed reductions proposed in Table 1 for narrower cross sections are proportional to those found in Table 6 for the less demanding road (N 222). Moreover, despite the differences in the baseline scenario between this study and the HCM (*1*), the speed reduction for the narrowest cross section observed at N 222 was close to the value proposed by the HCM (*1*): approximately 10 km/h.

The geometric characteristics of the road with a lower design speed (N 105-2) were significantly different from the characteristics of the roads considered by the HCM (*1*). The HCM considers two-lane highways with operating speeds between 70 and 110 km/h. However, the comparison of the two roads shows that the effects of the width of the cross section on the free-flow speed depend on the range of speed. The average travel speed observed in the DriS for N 105-2 was approximately 68% of the speed for N 222. This proportion is approximately the same observed for the speed reductions between both roads for each lane and shoulder width combination.

Another important conclusion from Table 6 is that the effects caused by variations in lane and shoulder widths were not cumulative: the speed reduction for a combined lane and shoulder decrease was higher than the sum of the speed reductions caused by the correspondent decreases in lane and shoulder widths individually. Marginal effects of lane and shoulder width on speed may be obtained through deriving the regression function (Equation 2) with respect to the shoulder [ln (s)] or lane [ln (l)] widths.

For example, in Table 6, it is possible to observe that the reduction in shoulder width from 0.8 m to 0.3 m varies according to the lane width. Likewise, the decrease in lane width from 3.6 m to 2.7 m depends on the shoulder's variation.

Conversely, the HCM (*1*) does not reflect different reductions in the free-flow speed from a combined lane and shoulder decrease or the sum of the correspondent individual effects (i.e., the speed reductions between two successive intervals of lane width presented in Table 1 are independent of shoulder width and vice versa).

Spot Speed Comparison Between Simulated and Real Environments

To validate the results from the driving simulator, a comparative analysis of spot speeds measured in the simulated and real environments was performed. For each of the six points of control, the average spot speeds in the real environment (SS_r) and the simulated environment (SS_s), the corresponding difference ($SS_s - SS_r$), and the standard deviations (SD_r and SD_s) were evaluated. The sample size in the real environment (N_r) depended on the traffic passing at each point of control. In the simulated environment, the sample size (N_s) was formed by the 15 drivers of the DriS.

Table 7 presents the variables used for comparing the spot speeds between the real and simulated environments. The radii and extensions of the road sections containing the points of control are also presented.

TABLE 3 Mean Values Collected for N 105-2

	Scenario							
	1	2	3	4	5	6	7	8
Lane width (m)	3	3	3.3	3.3	3.6	3.6	3	2.7
Shoulder width (m)	0.8	1.2	0.8	0.3	0.8	0.3	0.3	0.8
Platform width (m)	2 × 3.8	2 × 4.2	2 × 4.1	2 × 3.6	2 × 4.4	2 × 3.9	2 × 3.3	2 × 3.5
Average travel speed (km/h)	67.4	71.1	72.9	73.8	71.3	76.0	73.9	73.6
Number of errors								
Centerline	7	6	5	5	3	3	5	7
Sideline	10	11	13	13	3	3	1	0
Total	17	17	18	17	6	6	6	7
% errors								
Centerline	6.7	5.3	4.44	4.7	1.7	1.8	5.1	6.0
Sideline	13.1	12.7	18.9	19.0	2.1	2.6	0.4	0.0
Total	19.8	18.0	23.3	23.6	3.7	4.4	5.4	6.0
Average distance (m)								
Centerline	0.6	0.63	0.71	0.70	0.83	0.83	0.58	0.48
Sideline	0.49	0.46	0.38	0.38	0.86	0.85	1.10	1.20

TABLE 4 Mean Values Collected for N 222

	Scenario							
	1	2	3	4	5	6	7	8
Lane width (m)	3	3	3.3	3.3	3.6	3.6	3	2.7
Shoulder width (m)	1.8	2.2	1.8	1.2	1.8	1.2	1.2	1.8
Platform width (m)	2 × 4.8	2 × 5.2	2 × 5.1	2 × 4.5	2 × 5.4	2 × 4.8	2 × 4.2	2 × 4.5
Average travel speed (km/h)	106.4	118.3	121.7	123.2	105.80	116.4	115.9	116.2
Number of errors								
Centerline	1	1	1	1	1	1	1	2
Sideline	4	2	6	4	1	1	0	0
Total	5	3	7	5	2	2	1	2
% errors								
Centerline	1.3	1.8	1.5	1.7	1.0	0.8	1.2	3.7
Sideline	7.4	6.1	18.2	15.6	2.1	3.0	0.9	0.0
Total	8.7	8.0	19.7	17.4	3.1	3.9	2.0	3.7
Average distance (m)								
Centerline	0.62	0.61	0.75	0.72	0.90	0.89	0.65	0.50
Sideline	0.45	0.46	0.31	0.35	0.79	0.79	1.04	1.19

9	10	11	12	13	14	15	16
2.7	2.7	3	2.7	3.3	3.6	3.3	3.6
0.3	1.2	1.8	1.8	1.2	1.8	1.8	1.2
2 × 3	2 × 3.9	2 × 4.8	2 × 4.5	2 × 4.5	2 × 5.4	2 × 5.1	2 × 4.8
71.3	74.5	77.6	76.8	75.8	79.8	79.4	79.0
5	6	3	5	2	1	2	1
10	12	19	13	5	8	5	8
15	18	22	19	7	9	7	9
5.4	4.5	2.4	4.2	1.4	1.2	1.8	1.3
10.6	13.7	29.2	13.6	4.5	10.6	6.0	9.9
16.1	18.2	31.6	17.7	5.9	11.8	7.8	11.2
0.47	0.49	0.63	0.50	0.77	0.89	0.77	0.88
0.32	0.30	0.16	0.29	0.61	0.50	0.62	0.51

9	10	11	12	13	14	15	16
2.7	2.7	3	2.7	3.3	3.6	3.3	3.6
1.2	2.2	2.5	2.5	2.2	2.5	2.5	2.2
2 × 3.9	2 × 4.9	2 × 5.5	2 × 5.2	2 × 5.5	2 × 6.1	2 × 5.8	2 × 5.8
96.4	111.0	113.1	116.1	95.9	111.4	113.6	116.5
3	3	1	4	1	1	1	1
6	6	8	5	2	4	2	4
9	8	10	8	3	5	3	5
4.2	5.1	2.8	5.5	1.8	1.2	1.5	1.6
15.6	14.8	28.9	14.7	7.2	13.4	3.3	11.5
19.9	19.9	31.8	20.2	9.0	14.7	4.9	13.1
0.47	0.47	0.60	0.46	0.75	0.88	0.71	0.82
0.29	0.30	0.16	0.30	0.62	0.48	0.65	0.54

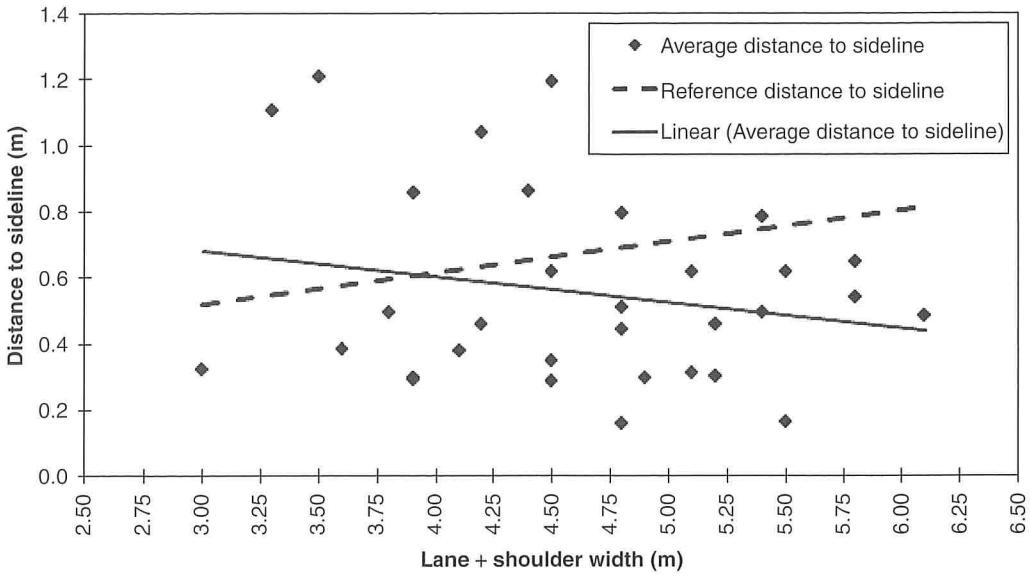


FIGURE 2 Average distance to sideline for different platform widths.

Despite the sample size differences between the real and simulated environments, the statistical dispersion observed for SS_r and SS_s is reasonably similar. On N 222, where characteristics other than geometry can produce higher effects on traffic flow, speed dispersion was smaller for the real environment. On N 105-2, where drivers are strongly conditioned by the geometry of the horizontal alignment, the differences in speed dispersion tended to be smaller.

For each point of control, the samples obtained in both the real and simulated environments were considered as normally distributed samples for a significance level of 5% through the Kolmogorov–Smirnov test, revealing similarities in drivers' behavior.

TABLE 5 Coefficients of Multiple Linear Regression

Variable	Coefficient	*P*-Value
Constant	4.455	.000
Cross section		
Lane width (l)	0.143	.006
Shoulder width (s)	0.171	.050
Lane width × shoulder width (ls)	−0.128	.093
Dummy variables		
Road type: d_r	0.385	.000
Driver 1 (d_{d1})	−0.038	.182
Driver 2 (d_{d2})	−0.081	.005
Driver 3 (d_{d3})	−0.049	.090
Driver 4 (d_{d4})	0.002	.934
Driver 5 (d_{d5})	−0.349	.000
Driver 6 (d_{d6})	−0.159	.000
Driver 7 (d_{d7})	−0.159	.000
Driver 8 (d_{d8})	−0.119	.000
Driver 9 (d_{d9})	−0.075	.009
Driver 10 (d_{d10})	−0.133	.000
Driver 11 (d_{d11})	−0.091	.002
Driver 12 (d_{d12})	−0.135	.000
Driver 13 (d_{d13})	−0.086	.003
Driver 14 (d_{d14})	−0.122	.000

NOTE: Observation: d_{di} expresses the behavior of driver i when compared with the base driver. $R = .89$; $R^2 = .79$.

Table 7 shows that ordering the points of control of each road by speed results in the same sequence for both environments. Drivers tended to adopt higher speeds in the simulated environment than in the real environment. However, this result seems to reflect a translation of the absolute speed values that does not affect the conclusions of this study. Therefore, the results obtained with the DriS were considered to be valid, both in terms of relative and absolute speed differences.

Higher speeds were expected in the simulated environment because of the lack of grip effects and dynamic behavior of the car body. On N 222, the speed difference between both environments was approximately 40 km/h. On N 105-2, this value was reduced to approximately 30 km/h because of the constraints imposed on drivers by the more demanding road alignment.

CONCLUSIONS

Previous studies have suggested that the width of road cross sections affect the free-flow speed chosen by drivers. The HCM (*1*) and the corresponding background study [NCHRP Project 3-55(3)] (*2*) take those effects into account by proposing values for speed reductions

TABLE 6 Reduction in Free-Flow Speed (km/h) According to Lane and Shoulder Widths

	Shoulder Width (m)								
	N 105-2				N 222				
Lane Width (m)	0.3	0.8	1.2	1.8	0.3	0.8	1.2	1.8	2.2
2.7	6.8	3.7	2.3	1.0	9.9	5.4	3.4	1.5	0.5
3.0	4.6	2.3	1.4	na	6.7	3.4	2.1	0.7	na
3.3	2.5	1.1	0.6	na	3.7	1.7	0.8	na	na
3.6	0.6	0.0	na	na	0.9	0.0	na	na	na

NOTE: na = not applicable.

TABLE 7 Spot Speed Comparison of Real and Simulated Environments

Point of Control (R; Ext.) (m)	SS_r (km/h)	SD_r (km/h)	SS_s (km/h)	SD_s (km/h)	N_r	N_s	$SS_s - SS_r$ (km/h)
N 105-2							
I (∞; 334)	54.2	16.1	92.7	16.3	164	15	38.7
II (150; 80)	46.8	13.2	77.1	9.9	133	15	30.3
III (∞; 473)	76.3	18.6	98.6	11.2	112	15	23.6
N 222							
I (545; 282)	71.5	16.3	113.7	17.6	323	15	42.2
II (∞; 240)	76.9	16.8	119.4	20.3	325	15	42.4
III (∞; 408)	76.6	13.7	115.7	19.7	330	15	38.7

NOTE: R = radius; ext. = extension.

caused by different lane and shoulder widths. This study also confirmed the effects of the road cross section on the free-flow speed; this confirmation is emphasized by the significance of lane and shoulder width revealed in the multivariate analysis.

However, the obtained results differ from the HCM (*1*) proposals on three main points. The base cross section, defined as the minimum combination of lane and shoulder widths above which a decrease in the free-flow speed is no longer observed, is formed by a lane 3.6 m wide and a shoulder 0.8 m wide instead of a lane 3.6 m wide and a shoulder 1.8 m wide, as proposed by the HCM (*1*). Consequently, the speed reductions determined for smaller cross sections also differed from the HCM (*1*). However, the obtained speed reduction for the narrowest cross section (a lane of 2.7 m and shoulder of 0.3 m) was approximately 10 km/h, which is close to the same value as that proposed by the HCM (*1*). Finally, an interaction between the effects of lane and shoulder width on free-flow speed was revealed, and they are not cumulative. Therefore, a simultaneous decrease in lane and shoulder widths produces a greater reduction in the free-flow speed than the sum of the same effects taken individually. This conclusion may contribute to a review of the lack of evidence assumed in the NCHRP Project 3-55(3) (*2*) for the existence of such interaction.

Despite being a widely adopted reference manual, the HCM (*1*) was developed for North American conditions, and its applicability to road networks with different geometric and environmental characteristics (e.g., roads in Portugal or other European countries) should be carefully considered.

From a comparison of the two considered roads, it is possible to conclude that the influence of a cross section on the free-flow speed also depends on the order of magnitude of the driving speeds.

The validity of the results obtained in the simulated environment confirms that driving simulators are reliable and flexible tools that may be used to create and study realistic traffic situations.

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The Operational Effects of Geometrics Committee peer-reviewed this paper.

Use of Both Centerline and Shoulder Rumble Strips on High-Speed Two-Lane Rural Roadways

Impact on Lateral Lane Position and Passing Maneuvers of Vehicles

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A before-and-after field evaluation assessed the impacts that centerline and shoulder rumble strips on rural two-lane roadways have on various characteristics of driver behavior. The following characteristics were assessed as part of this evaluation: passing maneuvers, lateral lane placement, and centerline and edgeline encroachments. The data were collected with video cameras temporarily installed at 18 passing zones and 12 curves on rural two-lane roadways throughout Michigan. The cameras were installed at the same locations before and after the installation of rumble strips. A manual review of the videos was performed to assess the various behavioral characteristics. Nearly 78,000 vehicles were observed during review of the passing zone videos, and more than 50,000 vehicles were observed during review of the curve videos. Centerline rumble strips were found to improve driver central lane positioning tendencies and decrease the occurrence of centerline encroachments. The inclusion of shoulder rumble strips in addition to centerline rumble strips provided incremental improvements in central lane positioning and reduced edgeline encroachments. Most notable, centerline and shoulder rumble strips greatly reduced the occurrence of drivers laterally shifting to the inside while maneuvering through curves (i.e., corner cutting). In addition, centerline rumble strips did not reduce the rate of passing maneuvers. Collectively, these results suggest that both centerline and shoulder rumble strips have a positive impact on surrogate measures for roadway safety and do not negatively affect driver behavior.

In 2008, approximately 23,067 crashes (129 fatal) occurred on the 5,700 mi of state-maintained, high-speed, nonfreeway road segments in Michigan (*1*). Among the most severe crashes are those that involve lane departure, which occurs when a vehicle crosses over the centerline or the edgeline. These crashes are often caused by driver distraction or drowsiness, resulting in a head-on collision, opposite direction sideswipe, or run-off-the-road collision. Lane departure crashes totaled 20.6% of all crashes and 61.2% of fatal crashes that occurred on these Michigan roadway segments in 2008.

In an effort to reduce the number of lane departure crashes, in 2008, the Michigan Department of Transportation (DOT) began a 3-year statewide initiative that called for the installation of milled centerline rumble strips on all rural nonfreeway highways with a posted speed limit of 55 mph and a paved roadway width greater than 20 ft (*2*). Milled shoulder rumble strips were also to be installed on roadways with shoulders that were at least 6 ft wide. This initiative was recognized as the largest nonfreeway rumble strip installation in the United States. The magnitude of this initiative will ultimately result in a wealth of data that can be used to assess the immediate and long-term impacts of centerline rumble strips (used alone or with shoulder rumble strips) on a variety of system-wide performance measures, especially safety, as well as secondary measures, including passing behavior, impacts on bicyclists, noise impacts, and pavement condition.

Previous research has provided substantial evidence that shoulder rumble strips (*3–7*) and centerline rumble strips (*7–9*) provide significant reductions in targeted lane departure crashes on two-lane roadways by as much as 15% and 30%, respectively (*7*). However, few comparisons have been made of the relative difference in safety performance of roadways with both centerline rumble strips and shoulder rumble strips and roadways with centerline rumble strips only. More broadly, few studies have focused on the impacts of rumble strips on salient driver behavior characteristics that have helped contribute to these crash reductions on two-lane roadways.

This research aims to gain important insight into these issues by assessing the impacts of centerline rumble strips on driver behavior characteristics related to lane departure crashes, including vehicular lateral placement within the travel lane, edgeline encroachments, and centerline encroachments (*10*). Such measures of effectiveness (MOEs) have been used to evaluate the impacts on driver behavior of rumble strips installed on rural undivided highways (*11, 12*). These MOEs have also been used to evaluate other lane departure treatments, such as post-mounted delineators, chevrons, wider and brighter pavement markings, and retroreflective raised pavement markers (*13–16*). Indication of any behavioral improvements would provide preliminary evidence of potential safety impacts and would provide insight into the changes in driver behavior that

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may have contributed to the crash reductions observed in previous research. Also of interest is the occurrence of potentially negative impacts induced by the rumble strips, such as reductions in passing attempts.

To evaluate the impacts of rumble strips on two-lane roadways, an intermediate evaluation was initiated in June 2010 before installation of the rumble strips in the Michigan DOT initiative was completed. The objectives of this evaluation were to assess the impact of centerline and shoulder rumble strips on

- Vehicular lateral placement within the travel lane,
- Vehicular encroachment onto or over the centerline or edgeline, and
- Attempted passing maneuvers.

FIELD STUDY

A before-and-after field study was performed to assess the impacts of centerline rumble strips and shoulder rumble strips on driver behavior along rural two-lane highways in Michigan. The study included both horizontal curves and tangent sections with and without passing zones. The following characteristics were assessed during the field study:

- Vehicular lateral placement in the travel lane,
- Encroachments onto or across the centerline,
- Encroachments onto or across the edgeline,
- Total passing attempts, and
- Aborted passing attempts.

Study Sites

Ten roadway segments were selected for the field evaluation. The segments were selected from the statewide population of two-lane rural highways with 55 mph speed limits on which rumble strips were scheduled for installation during summer 2010. The segments were evenly split between locations at which both centerline and shoulder rumble strips were installed and sites at which only centerline rumble strips were installed. The average daily traffic volumes at the 10 study segments ranged from 1,500 to 6,000 vehicles per day.

Before data collection, a preliminary investigation was performed along each roadway to identify at least one location at which passing was permitted in both directions of travel and one horizontal curve location (if present). From the 10 roadway segments, 18 passing zones and 12 horizontal curves were selected for data collection. The characteristics of the roadways used in this study, along with the number of specific data collection locations, are presented in Table 1.

Rumble Strip Installations

The rumble strips were installed at the study sites in late summer 2010. All roadways received centerline rumble strips, and roadways with shoulders that were at least 6 ft wide also received shoulder rumble strips. The rumble strip corrugations were ground (i.e., milled) into the pavement, per Michigan DOT specifications. Michigan DOT standard installation details (*17*) for both centerline rumble strips and shoulder rumble strips installations are shown in Figure 1 and summarized below.

- Centerline rumble strips:
  - Transverse dimension of corrugation (tolerance): 16 in. (±½ in.),
  - Longitudinal dimension of corrugation: 7 in. (±½ in.),
  - Spacing between corrugations: 5 in. (–½ in., +1 in.),
  - Longitudinal gap between corrugation pairs: 17 in., and
  - Depth of corrugation, outer edges: ⅜ in. (–0, +⅛ in.); center: ½ in. (–0, +⅛ in.) and
- Shoulder rumble strips:
  - Transverse dimension of corrugation (tolerance): 12 in. (±½ in.),
  - Longitudinal dimension of corrugation: 7 in. (±½ in.),
  - Spacing between corrugations: 5 in. (±½ in.),

TABLE 1 Site Characteristics

Rumble Strip Installed	Highway	Lane Width (ft)	Paved Shoulder Width (ft)	Average Daily Traffic (2009)	Number of Data Collection Locations	
Centerline and shoulder	M-19, Site 1	12	6	5,500	Passing zones	3
					Curves	2
	M-25	12	8	3,300	Passing zones	2
	M-136, Site 2	11	8	6,000	Passing zones	1
	US-41, Site 1	12	8	4,100	Passing zones	1
					Curves	1
	US-41, Site 2	12	8	4,500	Passing zones	1
					Curves	1
Centerline	M-19, Site 2	11	3	5,300	Passing zones	1
					Curves	3
	M-46	11	3	4,900	Passing zones	2
	M-136, Site 1	11	3	1,500	Passing zones	3
					Curves	2
	M-93	12	5	2,900	Passing zones	1
					Curves	2
	M-81	12	3	4,800	Passing zones	3
					Curves	1

NOTE: Rumble strips were not present at any location in before period.

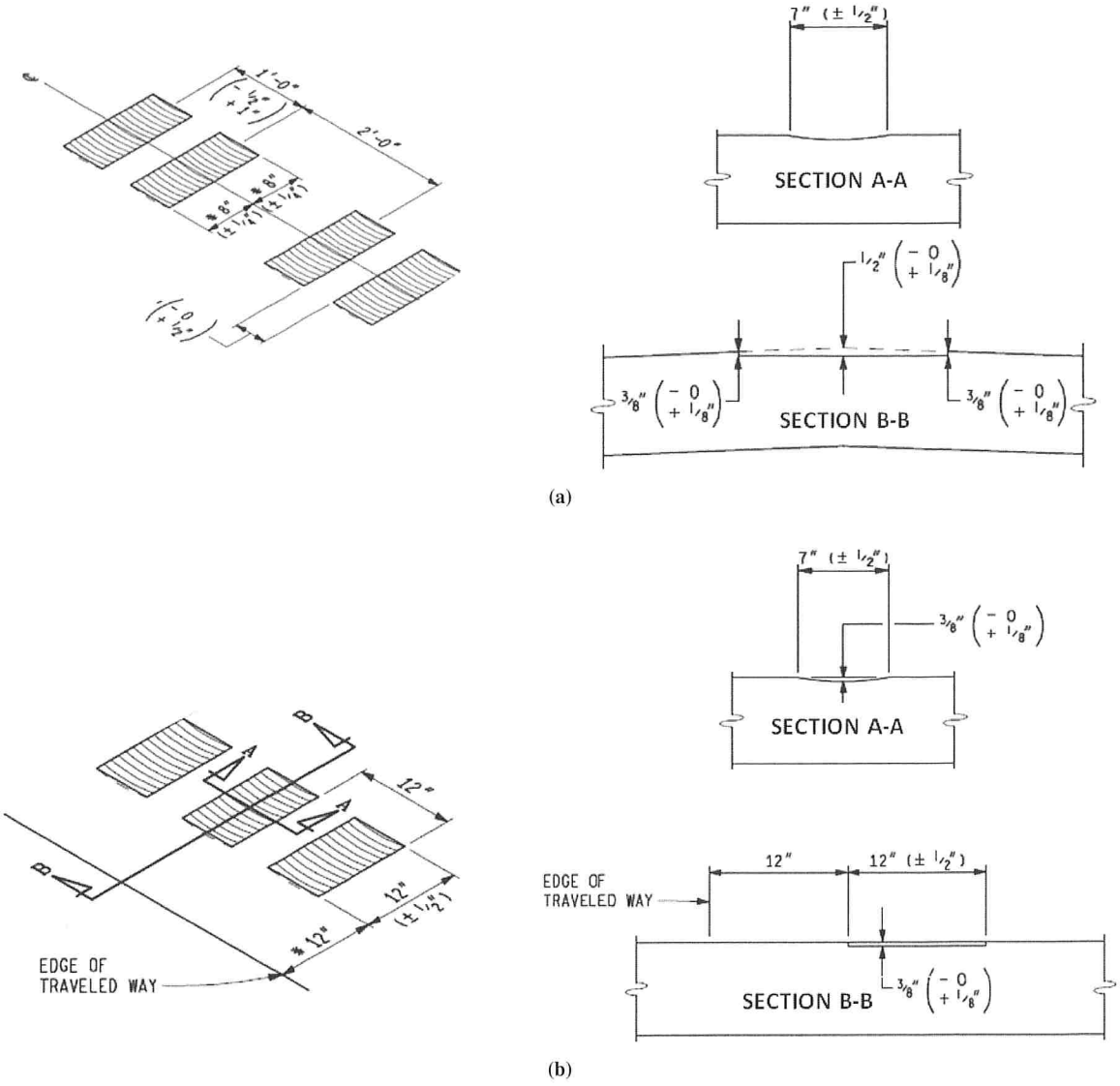


FIGURE 1 Michigan DOT rumble strip standards for rural nonfreeway roadways (*17*): (*a*) centerline rumble strips and (*b*) shoulder rumble strips.

– Offset from edge of traveled way to near edge of corrugation: 12 in.,

– Longitudinal installation cycle: 48 ft of rumble strips followed by a 12-ft gap, and

– Depth of corrugation: ⅜ in. (–0, +⅛ in.).

The rumble strip dimensions at each location were verified to comply with the tolerances of the Michigan DOT specification.

Data Collection

Video data were collected at the study sites before and after rumble strip installation. The before period data were collected between June 2010 and August 2010. Data were again collected in November 2010, May 2011, and June 2011 after the rumble strips had been in place for a minimum of 30 days. All data were collected during daylight hours under dry pavement conditions. Geometric data, including lane width, shoulder width, lateral offset of the rumble strips from the centerline or shoulder, and rumble strip dimensions, were measured at each data collection location.

Elevated high-definition video cameras were temporarily installed at each study site to covertly record the behavior of vehicles traveling through the study sites. Each camera was mounted on top of a lightweight aluminum pole that telescoped from 7 to 20 ft. The pole was securely strapped to a rigid roadside signpost. Between 4 and 10 h of video were typically recorded at each location during each data collection period.

A single camera setup was used at the curve locations and was mounted in a position that maximized the field of view of vehicles traveling through both the curve and the adjacent tangent section. The maximum clear viewing distance along a roadway for a single camera elevated to 20 ft was approximately 1,000 ft. The passing zone locations used two cameras mounted at the same location but aimed in opposite directions along the roadway. The two-camera setup doubled the effective viewing distance and greatly increased

the likelihood of capturing passing events in their entirety. Examples of the video camera setups for both passing zones and curves are shown in Figure 2.

Extraction of Behavioral Data

After the field data were collected, the videos were reviewed by a team of trained reviewers to assess various characteristics of driver behavior. QuickTime video players were used to review the videos. Each vehicle was monitored through the entire field of view of the camera or cameras. The behavioral characteristics that were assessed for each observed vehicle depended on whether the location was a passing zone or a horizontal curve.

Passing Zones

The videos recorded at the 18 passing zones were reviewed to assess numerous characteristics related to any passing maneuvers that were attempted by vehicles traveling through each study site. Synchronization of the time clocks between the two cameras used in each passing zone setup simplified the video review process by allowing for vehicles to be continuously tracked between the two videos. During the review of the dual-camera passing zone videos, several important characteristics were assessed for each vehicle traveling through the study site:

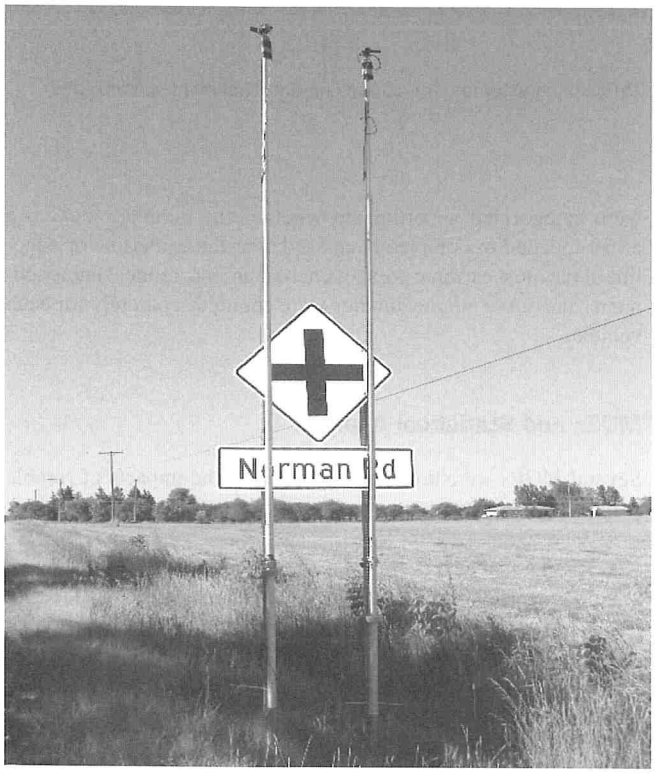
- Type of vehicle (passenger vehicle; truck, recreational vehicle, or bus; motorcycle),
- Travel direction,
- Position of the vehicle [within 150 ft of the previous vehicle (i.e., in passing position)],
- Attempted pass, and
- Aborted pass.

Vehicles were considered to be in a position to pass if they were within 150 ft of the previous vehicle. For vehicles traveling at 55 mph, a 150-ft following distance represents a headway of approximately 2 s between the leading vehicle and the following vehicle. The distance between successive vehicles was assessed on the basis of the number of centerline skip pavement markings, which were installed at 50-ft intervals per the Michigan DOT standard.

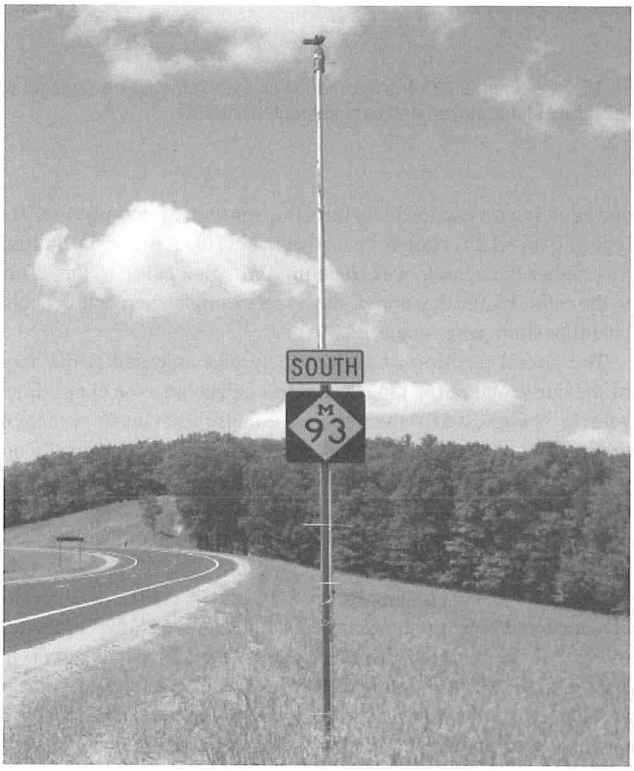
A passing attempt was defined as a vehicle that crossed the centerline and began to overtake another vehicle that was traveling in the same lane. Aborted passing attempts were defined as cases in which a vehicle initially touched or crossed the centerline while attempting to overtake another vehicle, but moved back into the original position without completing the pass. It was not possible to distinguish and subsequently exclude unintentional shifts in lateral position that resulted in a contact with the centerline. Figure 3, *a* and *b*, shows an example of the vehicular assessments performed during review of the passing zone videos.

Curves and Adjacent Tangent Sections

The videos recorded at the 12 curves were reviewed to assess the lateral lane position and encroachments onto or over the centerline



(a)



(b)

FIGURE 2 Typical elevated video camera setup: (*a*) passing zone (two cameras in opposing directions) and (*b*) curve (single camera aimed toward curve).

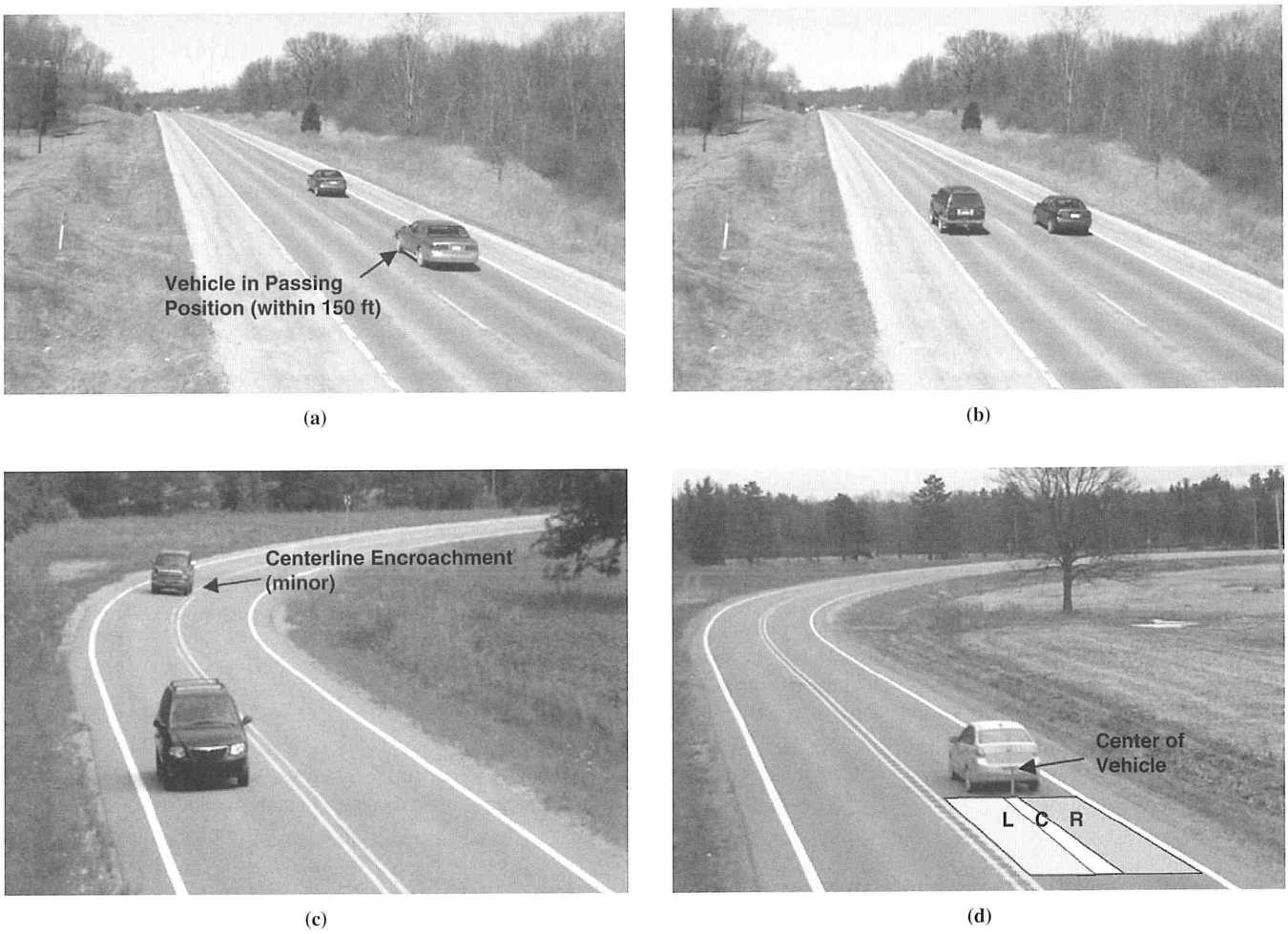


FIGURE 3 Examples of driver behavior assessment: (*a*) passing position, (*b*) passing attempt, (*c*) center line encroachment (curve), and (*d*) lateral lane placement assessment (tangent).

and edgeline for each vehicle traveling through each study site. The type and travel direction for each vehicle were recorded, in addition to whether the vehicle was traveling through a curve to the left or to the right. Figure 3, *c* and *d,* shows an example encroachment and lateral position assessment.

The lateral position of each vehicle was assessed at the apex of the curve and at the tangent section adjacent to the curve. Each vehicle was assessed at the same location for each curve or tangent section for both the before and after periods. It was occasionally not possible to assess the lateral position of a vehicle in both the curve and adjacent tangent. As a result, there is a slight imbalance between the number of observations for the curve and tangent data sets.

The lateral position was assessed according to the center of the vehicle with respect to the center of the lane. A vehicle was considered centered unless the center of the vehicle had shifted to the left or right of the center of the lane by more than approximately 6 in. The vehicle's license plate was often used as a reference point to assess the lateral position. This procedure was used previously by the authors in an evaluation of vehicular lateral placement in work zones (*18*).

Each vehicle was also monitored to determine whether a centerline or edgeline encroachment occurred at any point along the visible portion of the tangent section or curve section. Encroachments were categorized according to whether the vehicle's near tire either touched or completely crossed over the centerline or edgeline at the most extreme point of encroachment. Tangent encroachments and curve encroachments were counted separately for each vehicle.

MOEs and Statistical Analysis

Several MOEs were developed to quantify the impacts of rumble strips on driver behavior.

- Passing maneuvers:
  - –Percentage of vehicles that attempted a passing maneuver,
  - –Percentage of vehicles that were in a position to pass that attempted a passing maneuver, and
  - –Percentage of vehicles that aborted a passing maneuver after an initial attempt;
- Lateral position within travel lane:
  - –Percentage of vehicles centered in the lane,
  - –Percentage of vehicles in the right lane position, and
  - –Percentage of vehicles in the left lane position; and
- Encroachments:

– Percentage of vehicles encroaching onto or across the centerline and

– Percentage of vehicles encroaching onto or across the edgeline.

Each MOE was expressed as dichotomous rates of occurrence, and as such, two sample z-tests of proportions were used to determine the statistical significance of changes in the MOEs between the before and after periods. Two-tailed tests were used for all statistical testing, and the null hypothesis for all tests was that the rumble strips produced no change in the MOE. The lateral position and encroachment MOEs were analyzed both separately by vehicle type and overall for curves to the left, curves to the right, and tangent sections. The MOEs related to passing maneuvers were analyzed separately overall and by site.

Because several hypothesis tests were performed simultaneously on the same family of data for each MOE, it was necessary to apply a multiple comparison correction to correct for errors in inference that may occur (*19*). The Bonferroni multiple comparison correction was applied to the analyses to correct for erroneous rejection of the null hypothesis that is typically encountered during individual testing of several hypotheses from the same family of data. The Bonferroni correction assumes the selected significance level, $100 - \alpha$ (percent), to relate to inference on the family of data, where α is the selected probability of Type 1 error for the entire family of data. The corresponding significance level used for each individual hypothesis test is equal to $(100 - \alpha)/n$ (percent), where n is the number of simultaneous tests being performed per MOE (e.g., one test for each study location plus one overall test). Critical z-values (or t-values) for rejection of the null hypothesis were determined accordingly from the standard normal probability table.

DISCUSSION OF RESULTS

Passing Maneuvers

The review of the passing zone videos yielded 39,664 and 38,094 vehicles in the before and after periods, respectively. A total of 1,188 passing attempts were observed, 620 during the before period and 568 during the after period. Twenty-seven of these passing attempts were aborted, including 14 in the before period and 13 in the after period. The descriptive statistics resulting from the review of the passing zone videos are presented in Table 2.

A summary of the statistical results for the MOEs related to passing maneuvers is provided in Table 3. The overall percentage of vehicles attempting a passing maneuver decreased slightly from 1.56% to 1.49% after the rumble strips were installed, although this difference was not statistically significant. The site-by-site analysis found 17 of the 18 study locations to have no significant change in the rate of passing attempts.

As shown in Table 3, the total passing attempts did not change significantly when analyzed as a percentage of vehicles in a position to pass. The percentage of vehicles that were in a position to pass and attempted a passing maneuver increased slightly from 9.86% to 10.55%, which was not statistically significant. Fifteen of the 18 study locations showed no significant changes in passing attempts as a percentage of total vehicles in a position to pass.

Similarly, no statistically significant changes were found in the rate of aborted passing attempts. Overall, passing maneuvers were aborted in 2.26% of all passing attempts before rumble strip installation and 2.29% of all passing attempts after rumble strip installation. The site-by-site analysis also showed no statistically significant changes in the rate of aborted passing attempts at any of the study sites.

TABLE 2 Descriptive Statistics for Passing Maneuvers

	Total Observation Time (h)		Total Vehicle Count		Vehicles in Passing Position		Total Passing Attempts		Aborted Passing Attempts	
Data Collection Location	Before	After	Before	After	Before	After	Before	After	Before	After
M-136, Site 1, PZ 1	5.0	9.3	588	1,103	56	79	19	21	0	0
M-136, Site 1, PZ 2	6.4	5.5	2,657	1,581	392	158	23	18	0	2
M-136, Site 1, PZ 3	5.9	8.6	767	1,193	52	71	10	15	0	0
M-136, Site 2, PZ 1	5.1	7.6	2,403	3,527	460	565	22	18	2	0
M-19, Site 1, PZ 1	4.7	8.6	1,926	2,684	375	456	19	30	0	1
M-19, Site 1, PZ 2	4.6	7.8	1,811	2,513	299	419	25	38	1	2
M-19, Site 1, PZ 3	8.4	8.6	3,037	2,636	542	438	57	57	4	2
M-19, Site 2, PZ 1	6.2	8.8	1,920	3,016	373	427	42	34	1	2
US-41, Site 1, PZ 1	6.5	10.0	1,661	2,508	197	318	7	21	0	1
US-41, Site 2, PZ 1	3.9	7.9	1,011	2,498	138	393	13	36	0	1
M-93, PZ 1	8.2	9.3	1,935	1,835	162	133	15	21	0	1
M-46, PZ 1	9.4	2.2	3,258	608	545	91	67	13	0	0
M-46, PZ 2	9.0	7.1	3,166	2,352	445	297	20	15	0	0
M-25, PZ 1	6.4	8.3	2,436	2,150	408	321	41	60	1	0
M-25, PZ 2	5.3	8.6	2,730	2,530	553	356	34	43	0	1
M-81, PZ 1	8.7	7.4	2,728	1,915	457	331	113	46	2	0
M-81, PZ 2	8.4	2.1	3,151	653	484	65	48	6	2	0
M-81, PZ 3	7.3	8.4	2,479	2,782	353	464	45	76	1	0
Total	119.4	136.0	39,664	38,084	6,291	5,382	620	568	14	13

NOTE: PZ = passing zone.

TABLE 3 Statistical Analysis Results for Passing-Related MOEs

Data Collection Location	Total Passing Attempts (% of Total Vehicles)			Total Passing Attempts (% of Vehicles in Passing Position)			Aborted Passing Attempts (% of Total Passing Attempts)		
	Before	After	Significant Difference?	Before	After	Significant Difference?	Before	After	Significant Difference?
M-136, Site 1, PZ 1	3.23	1.90	No	33.93	26.58	No	0.00	0.00	No
M-136, Site 1, PZ 2	0.87	1.14	No	5.87	11.39	No	0.00	11.11	No
M-136, Site 1, PZ 3	1.30	1.26	No	19.23	21.13	No	0.00	0.00	No
M-136, Site 2, PZ 1	0.92	0.51	No	4.78	3.19	No	9.09	0.00	No
M-19, Site 1, PZ 1	0.99	1.12	No	5.07	6.58	No	0.00	3.33	No
M-19, Site 1, PZ 2	1.38	1.51	No	8.36	9.07	No	4.00	5.26	No
M-19, Site 1, PZ 3	1.88	2.16	No	10.52	13.01	No	7.02	3.51	No
M-19, Site 2, PZ 1	2.19	1.13	No	11.26	7.96	No	2.38	5.88	No
US-41, Site 1, PZ 1	0.42	0.84	No	3.55	6.60	No	0.00	4.76	No
US-41, Site 2, PZ 1	1.29	1.44	No	9.42	9.16	No	0.00	2.78	No
M-93, PZ 1	0.78	1.14	No	9.26	15.79	No	0.00	4.76	No
M-46, PZ 1	2.06	2.14	No	12.29	14.29	No	0.00	0.00	No
M-46, PZ 2	0.63	0.64	No	4.49	5.05	No	0.00	0.00	No
M-25, PZ 1	1.68	2.79	No	10.05	18.69	Yes	2.44	0.00	No
M-25, PZ 2	1.25	1.70	No	6.15	12.08	Yes	0.00	2.33	No
M-81, PZ 1	4.14	2.40	Yes	24.73	13.90	Yes	1.77	0.00	No
M-81, PZ 2	1.52	0.92	No	9.92	9.23	No	4.17	0.00	No
M-81, PZ 3	1.82	2.73	No	12.75	16.38	No	2.22	0.00	No
Total	1.56	1.49	No	9.86	10.55	No	2.26	2.29	No

NOTE: Statistically significant at 95% confidence level on the basis of a Bonferroni corrected critical z-score of ±3.00.

Lateral Lane Position

The review of the videos from the curves yielded 30,202 and 20,673 vehicles in the before and after periods, respectively. The lateral lane position data were aggregated according to the types of rumble strips installed, geometry, and vehicle type. Site-by-site analyses were not performed. The results of the vehicular lateral lane position analysis are presented in Tables 4 and 5 for locations with centerline rumble strips only and locations with centerline rumble strips and shoulder rumble strips, respectively.

The presence of rumble strips had a statistically significant impact on the lateral lane position of vehicles in both curve and tangent sections. Vehicles tended to be more centrally positioned within the lane when rumble strips were present because drivers tended to shy away from both the centerline and the edgeline in the presence of rumble strips. This result was especially evident for locations with both centerline and shoulder rumble strips because the percentage of vehicles positioned in the center of the lane approximately doubled in both curve and tangent sections after rumble strip installation. Although central lane positioning increased after rumble strip installation for locations with centerline rumble strips only, the increases were of a lower magnitude and less consistent compared with locations with both centerline and shoulder rumble strips.

The results were found to vary somewhat according to vehicle type. Both passenger vehicles and large vehicles, such as trucks, buses, and recreational vehicles, showed significant increases in center lane positioning when rumble strips were present, especially at locations at which both centerline and shoulder rumble strips were present. Large vehicles showed the greatest changes in lateral position when rumble strips were present, especially on curves to the right. The percentage of vehicles positioned in the center doubled at locations at which only centerline rumble strips were installed and nearly tripled where both centerline and shoulder rumble strips were installed. The central lane positioning of motorcycles was improved by the presence of rumble strips at locations at which both centerline and shoulder rumble strips were installed. The presence of centerline rumble strips alone did not significantly affect the lane position of motorcycles.

Encroachments

Centerline and edgeline encroachments were also assessed within the curve and along the adjacent tangent section for each vehicle observed in the videos. Only locations at which both centerline and shoulder rumble strips were installed were included in the assessment of edgeline encroachments. Similar to the lateral lane position data, the encroachment data were aggregated according to geometry and vehicle type. The results of the encroachment analysis are presented in Table 6.

The presence of rumble strips had a statistically significant reduction in both centerline and edgeline encroachments in both curve and tangent sections. The greatest reductions in centerline encroachments were observed within curves to the left; encroachments were reduced by 87.5% from 11.9% to 1.5%. Similarly, the greatest reductions in edgeline encroachments were observed within curves to the right; encroachments were reduced by 43.7% from 11.6% to

TABLE 4 Vehicular Lateral Lane Position Results by Geometry and Type of Vehicle: Locations with Centerline Rumble Strips Only

	Total Volume		Left of Center (%)			Centered in Lane (%)			Right of Center (%)		
Vehicle Type	Before	After	Before	After	Change	Before	After	Change	Before	After	Change
Tangents											
Passenger	19,499	11,749	22.1	18.5	−16.3[a]	36.5	48.8	33.8[a]	41.4	32.7	−21.0[a]
Truck, bus, RV	996	603	17.0	15.4	−9.1	33.6	44.1	31.2[a]	49.4	40.5	−18.1[a]
Motorcycle	384	143	42.7	41.3	−3.4	34.1	32.2	−5.7	23.2	26.6	14.7
All	20,879	12,495	22.3	18.6	−16.3[a]	36.3	48.4	33.3[a]	41.4	33.0	−20.4[a]
Left Curves											
Passenger	11,327	6,489	41.1	19.0	−53.7[a]	33.0	55.7	68.5[a]	25.8	25.3	−2.2
Truck, bus, RV	560	348	31.3	21.3	−32.0[a]	33.8	47.1	39.6[a]	35.0	31.6	−9.7
Motorcycle	219	82	51.1	41.5	−18.9	35.2	28.0	−20.2	13.7	30.5	122.6[a]
All	12,106	6,919	40.8	19.4	−52.5[a]	33.1	54.9	65.9[a]	26.1	25.7	−1.5
Right Curves											
Passenger	8,175	5,230	6.1	6.7	9.3	24.5	45.4	85.1[a]	69.4	47.9	−30.9[a]
Truck, bus, RV	434	259	5.3	11.2	111.3	23.0	45.6	97.7[a]	71.7	43.2	−39.7[a]
Motorcycle	165	57	18.2	26.3	44.7	37.0	40.4	9.1	44.8	33.3	−25.7
All	8,774	5,546	6.3	7.1	12.6	24.7	45.3	83.7[a]	69.0	47.6	−31.1[a]

NOTE: The before-and-after percent change was computed as follows: [(A − B)/B] × 100%. RV = recreational vehicle.
[a]Statistically significant at 95% confidence level on the basis of a Bonferroni corrected critical z-score of ±2.86.

6.6%. These findings suggest that rumble strips tend to reduce the tendency for drivers to laterally shift to the inside while maneuvering through curves (i.e., cut corners). Both centerline and edgeline encroachments were also reduced in tangent sections, although the changes were less extreme than those within the curve sections.

The encroachment results varied according to vehicle type. Passenger vehicles showed consistent and significant reductions in both centerline and edgeline encroachments after the installation of rumble strips for nearly all geometric conditions. Large vehicles showed mostly marginal decreases in encroachments after the rumble strips were installed, although centerline encroachments were significantly reduced on curves to the left. Encroachments by motorcycles onto the centerline and particularly the edgeline were rare and were not significantly affected by the presence of rumble strips.

TABLE 5 Vehicular Lateral Lane Position Results by Geometry and Type of Vehicle: Locations with Centerline Rumble Strips and Shoulder Rumble Strips

	Total Volume		Left of Center (%)			Centered in Lane (%)			Right of Center (%)		
Vehicle Type	Before	After	Before	After	Change	Before	After	Change	Before	After	Change
Tangents											
Passenger	8,567	7,560	32.8	9.7	−70.3[a]	34.9	68.5	96.6[a]	32.4	21.7	−32.9[a]
Truck, bus, RV	603	559	30.2	7.0	−76.9[a]	35.7	71.0	99.2[a]	34.2	22.0	−35.6[a]
Motorcycle	145	59	49.0	20.3	−58.5[a]	35.9	72.9	103.2[a]	15.2	6.8	−55.3
All	9,315	8,178	32.9	9.6	−70.7[a]	34.9	68.7	96.8[a]	32.2	21.6	−32.9[a]
Left Curves											
Passenger	5,516	4,644	19.9	4.4	−78.0[a]	33.9	72.5	113.7[a]	46.1	23.1	−49.9[a]
Truck, bus, RV	375	337	14.1	3.6	−74.8[a]	32.0	73.9	130.9[a]	53.9	22.6	−58.1[a]
Motorcycle	110	38	42.7	28.9	−32.3	30.9	63.2	104.3[a]	26.4	7.9	−70.1
All	6,001	5,019	20.0	4.5	−77.4[a]	33.8	72.5	114.9[a]	46.2	22.9	−50.4[a]
Right Curves											
Passenger	3,055	2,915	20.3	1.9	−90.7[a]	35.1	66.8	90.3[a]	44.6	31.3	−29.7[a]
Truck, bus, RV	227	208	37.4	0.5	−98.7[a]	26.4	75.0	183.8[a]	36.1	24.5	−32.1
Motorcycle	39	21	17.9	4.8	−73.5	46.2	85.7	85.7[a]	35.9	9.5	−73.5
All	3,321	3,144	21.5	1.8	−91.6[a]	34.6	67.5	94.8[a]	43.9	30.7	−30.0[a]

NOTE: The before-and-after percent change was computed as follows: [(A − B)/B] × 100%.
[a]Statistically significant at 95% confidence level on the basis of a Bonferroni corrected critical z-score of ±2.86.

TABLE 6 Encroachment Results by Geometry and Type of Vehicle

Vehicle Type	% Encroaching onto or Across Edgeline			% Encroaching onto or Across Centerline		
	Before	After	% Change	Before	After	% Change
Tangents						
Passenger	9.1	5.4	−41.2[a]	1.5	0.6	−63.7[a]
Truck, bus, RV	27.7	31.0	11.8	2.0	1.4	−31.2
Motorcycle	0.0	0.0	0.0	0.8	1.0	30.9
All	10.5	6.6	−37.1[a]	1.5	0.6	−60.7[a]
Left Curves						
Passenger	11.2	3.7	−67.2[a]	12.0	1.3	−88.8[a]
Truck, bus, RV	36.7	26.5	−27.9	13.6	4.1	−69.9[a]
Motorcycle	0.0	0.0	0.0	3.3	0.0	−100.0
All	13.2	4.5	−65.7[a]	11.9	1.5	−87.5[a]
Right Curves						
Passenger	10.3	5.4	−47.5[a]	0.5	0.4	−28.6
Truck, bus, RV	28.8	27.1	−6.0	1.8	1.1	−41.0
Motorcycle	0.0	0.0	0.0	0.5	0.0	−100.0
All	11.6	6.6	−43.7[a]	0.6[a]	0.4	−31.4

NOTE: Only locations where shoulder rumble strips were installed between the before-and-after periods were included in the assessment of edgeline encroachments. The before-and-after percent change was computed as follows: $[(A - B)/B] \times 100\%$.
[a]Statistically significant at 95% confidence level on the basis of a Bonferroni corrected critical z-score of ±2.86.

A subsequent analysis for cases in which at least one wheel was completely across the centerline or edgeline (i.e., major encroachment) was also performed. Major encroachments across the edgeline were relatively infrequent events and were not significantly affected by the presence of rumble strips under any geometric condition, although marginal decreases were observed. Major encroachments across the centerline decreased significantly after the installation of rumble strips for both tangent sections and curves to the left. Major centerline encroachments were not affected by rumble strips for curves to the right.

CONCLUSIONS

This paper presents the results of a before-and-after field study performed to assess whether the installation of centerline and shoulder rumble strips on rural two-lane roadways affects driver behavior. Video recordings of driver behavior were obtained at 18 passing zones and 12 curves along 10 roadway segments before and after the installation of rumble strips. The following behavioral characteristics were assessed as part of this evaluation: passing maneuvers, lateral lane placement, and centerline and edgeline encroachments. The conclusions are summarized below.

- Passing attempts:
  - The presence of centerline rumble strips did not have an effect on the percentage of vehicles attempting to pass.
  - Aborted passing attempts were also not affected by centerline rumble strips.
  - These findings were similar to previous research that found little impact on passing maneuvers for locations with centerline rumble strips (*11*).
- Lateral lane position:
  - Vehicles tended to maintain a more centralized lateral lane position when centerline rumble strips were present. This finding is consistent with previous research, which found a general tendency for drivers to shift away from the centerline in the presence of centerline rumble strips (*11, 12*).
  - The increases in center lane positioning after rumble strip installation were more pronounced at locations at which both centerline and shoulder rumble strips were installed.
  - The changes in lateral lane position were similar for both tangent sections and curve sections, although slightly greater increases in center lane positioning were observed on curves.
  - Large vehicles, including trucks, recreational vehicles, and buses, showed the greatest increase in center lane positioning when rumble strips were present.
  - The center lane positioning of motorcycles was improved by the presence of rumble strips only at locations at which both centerline and shoulder rumble strips were installed. The presence of centerline rumble strips alone did not significantly affect the lane position of motorcycles.
- Encroachments:
  - Centerline and shoulder rumble strips greatly reduced the occurrence of drivers laterally shifting to the inside while maneuvering through curves (i.e., corner cutting).
  - The greatest reductions in centerline encroachments were observed for vehicles traveling through curves to the left.
  - The greatest reductions in edgeline encroachments were observed for vehicles traveling through curves to the right.
  - Centerline and edgeline encroachments were also reduced in tangent sections, although the changes were less extreme than in curve sections.

–Passenger vehicles showed consistent reductions in both centerline and edgeline encroachments for nearly all geometric conditions.

–Large vehicles showed mostly marginal decreases in encroachments after the rumble strips were installed, although centerline encroachments were significantly reduced on curves to the left.

–Encroachments by motorcycles were rare and not significantly affected by the presence of rumble strips.

In summary, centerline rumble strips improved drivers' central lane positioning tendencies and decreased the occurrence of centerline encroachments on two-lane roadways. Installation of shoulder rumble strips in addition to centerline rumble strips provided even greater improvements to central lane positioning and also decreased edgeline encroachments. In addition, the installation of centerline rumble strips did not negatively affect passing maneuvers. Collectively, these results suggest that both centerline and shoulder rumble strips have a positive impact on surrogate measures for lane departure crashes. There was little evidence of negative behavioral impacts. Applied broadly, such behavioral improvements have likely contributed to the reductions observed in previous research of lane departure crashes on two-lane roadways.

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The opinions, findings, and conclusions expressed are those of the authors and not necessarily those of the Michigan State Transportation Commission, the Michigan Department of Transportation, or FHWA.

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Super 2 Highways in Texas

Operational and Safety Characteristics

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As traffic volumes increase in both urban and rural areas, so do demands on the highway network. Specifically, as rural traffic volumes rise in Texas, the pressure on the state's network of two-lane highways rises accordingly. Previous research in Texas demonstrated that periodic passing lanes can improve operations on two-lane highways with average daily traffic lower than 5,000 vehicles. These highways, called Super 2 highways, can provide many of the benefits of a four-lane alignment at a lower cost. A recent project expanded on that research to develop design guidelines for passing lanes on two-lane highways with higher volumes. Researchers investigated the effects of volume, terrain, and heavy vehicles on traffic flow and safety. This paper discusses findings from field observations and crash analysis of existing Super 2 highway corridors in Texas and computer modeling of traffic conditions on a simulated Super 2 corridor. Results indicate that passing lanes provide added benefit at higher traffic volumes by reducing crashes, delay, and percent time spent following. Empirical Bayes analysis of crash data reveals a 35% reduction in expected nonintersection crashes with injuries. Simulation results indicate that most passing activity takes place within the first mile of the passing lane, so additional passing lanes can offer greater benefit than longer passing lanes. Whether new passing lanes are added or existing lanes are lengthened, the incremental benefit diminishes as additional length is provided and the highway more closely resembles a four-lane alignment.

Traditionally, roadway agencies expand a two-lane highway to four lanes when certain criteria are met, such as average daily traffic (ADT), peak volumes, prevailing speeds, or crash history. As more rural highways approach conditions that meet these criteria, agencies are looking for alternatives to full four-lane expansion to provide operational benefits at lower cost. Previous research demonstrated that periodic passing lanes can improve operations on two-lane highway corridors with low to moderate volumes (*1*). In Texas, a Super 2 highway is defined as a two-lane rural highway in which periodic passing lanes have been added to allow for the passing of slower vehicles and the dispersal of traffic platoons. The passing lane alternates from one direction of travel to the other within a section of roadway, allowing passing opportunities in both directions. These improved corridors can provide many of the benefits of a four-lane alignment at lower cost.

As traffic volumes, including volumes of heavy vehicles, increase on the state's two-lane roads, the effects of limited passing sight distance are magnified. The result is more locations where Super 2 highways can be effective, so providing longer passing lanes or providing passing lanes with greater frequency may be justified.

A recent project expanded on previous research to develop design guidelines for passing lanes on two-lane highways with higher volumes (*2*). Recommended guidelines from that project, submitted for inclusion in the Texas *Roadway Design Manual* (RDM), address geometric design criteria; placement, length, and spacing of passing lanes; and appropriate transitions at either end of passing lanes. This paper discusses findings from field observations and microscopic simulation, presents conclusions on operational characteristics, and summarizes analyses of data from crashes on Super 2 highways to develop conclusions about safety.

RECENT RESEARCH

Basis for Texas Guidelines

A previous research project produced recommendations for design guidelines for Super 2 highways in Texas (*1*). Researchers on that project collected field data at existing Super 2 corridors in Minnesota and Kansas to gain firsthand knowledge of operations and to personally view existing installations; they collected data on operating speeds, distribution of trucks, lane splits, and headways. Additional field studies at passing lane transitions in Texas provided data for comparison with the Minnesota and Kansas data. The research team also surveyed Texas drivers to gather their input and gauge drivers' attitudes toward passing behavior. In addition, researchers created a test bed scenario for microscopic simulation. They evaluated operating characteristics for a variety of passing lane lengths and spacing, traffic volumes, and heavy vehicle percentages. From their findings, researchers developed recommendations for passing lane length and spacing, lane and shoulder widths, signs, and pavement markings. The researchers' recommendations for length and spacing, shown in Table 1, were the basis for the guidelines in the October 2006 Texas RDM (*3*).

The RDM guideline values were similar to those recommended in Table 1, except that the RDM reduced the number of ADT categories and simplified the length and spacing ranges. The signing and marking layout developed in the research (*1*) identified two specific informational signs for the length and spacing and a dotted extension line marking to reinforce the preferred behavior that drivers should travel in the right lane except when passing.

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TABLE 1 Recommended Values of Length and Spacing by ADT and Terrain (*1*)

ADT (vpd)			
Level Terrain	Rolling Terrain	Recommended Passing Lane Length (mi)	Recommended Distance Between Passing Lanes (mi)
≤1,950	≤1,650	0.8–1.1	9.0–11.0
2,800	2,350	0.8–1.1	4.0–5.0
3,150	2,650	1.2–1.5	3.8–4.5
3,550	3,000	1.5–2.0	3.5–4.0

NOTE: 1 mi = 1.6 km; vpd = vehicles per day.

Traffic Volumes

The Texas research project provided recommendations for passing lanes on highways with no more than 5,000 vehicles per day (vpd); 2006 Texas Department of Transportation (DOT) RDM guidelines limited Super 2 recommendations to highways with less than 3,550 ADT, with the advice that a four-lane cross section should be considered for higher volumes. Recent studies have evaluated operations on higher-volume passing lane sections in other states.

In 2006, Gattis et al. reported on a study of passing lane operations; they focused on continuously alternating passing lane sections in Arkansas (*4*). Their field study contained four sites with average flow rates from 164 to 445 vehicles per hour and maximum flow rates from 232 to 724 vehicles per hour. Their findings indicated that the passing lane sections reduced the percentage of vehicles in platoons by about 14%, with much of that reduction coming in the first 0.9 mi (1.4 km) of the passing lane. They also found that passing maneuvers increased as volume increased, inferring that higher-volume roads could use longer passing lanes. A broader review of crash data at 19 passing lane sites showed that even though the average ADT of those sites (5,293 vpd) was almost three times the statewide average for rural two-lane undivided highways (1,857 vpd), the crash rates at 16 of those sites were lower than the statewide average of 1.4 crashes per million vehicle miles.

Potts and Harwood evaluated the benefits of passing lanes in Missouri (*5*). They analyzed three roadway sections and compared traffic operations before and after installation of passing lanes on rural two-lane highways. The three sites had ADTs ranging from 4,500 to 10,600 vpd, each with truck and recreational vehicle proportions of 10% and 5%, respectively. The analysis showed that the level of service (LOS) improved noticeably at each site, on the basis of the average travel speed and the percent time spent following; two of the three sites improved LOS by two grades. A review of crash data showed that the crash rates for two-lane highways with passing lanes were approximately 29% lower than the rates for traditional two-lane highways in the same districts.

Passing Lane Design Characteristics

Gattis et al. reported that the greatest benefits of passing lanes in their study of continuous three-lane cross sections (see Figure 1) were observed in the first 0.9 mi (1.4 km) (*4*). Between 0.9 and 1.9 mi (1.4 and 3.1 km), the benefits were less pronounced but were more likely to accrue as volumes increased. Where continuous three-lane cross sections with alternating passing lanes segments are present,

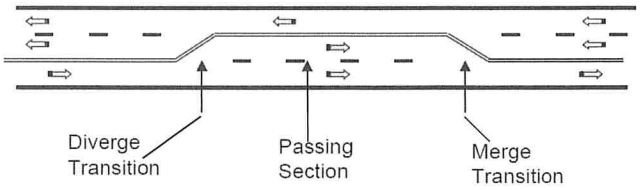

FIGURE 1 Schematic example of three-lane alternate passing design in Arkansas (*4*).

they concluded, agencies should reexamine the need for any passing lane that continues beyond approximately 1.9 mi (3.1 km) in length. This study suggests that a rather high volume is needed before extra length produces any notable degree of extra benefits. It may be that the other direction of travel would benefit more from an earlier termination of the passing lane and a switch in the direction of the lane.

EVALUATION OF EFFECTIVENESS

Existing Guidance on Evaluating Super 2 Performance

The *Highway Capacity Manual* (HCM) provides guidance on the evaluation and analysis of existing passing lane sections on the basis of microscopic simulation, field data, and theoretical concepts (*6*). According to the HCM, the capacity of a two-lane highway is 1,700 passenger cars per hour for each direction of travel, with a combined capacity of 3,200 passenger cars per hour in both directions for extended lengths of highway. However, these theoretical capacities and corresponding LOS are negatively affected by terrain, heavy vehicles, peak hours, lane and shoulder widths, and other factors. Providing a passing lane on a two-lane highway in level or rolling terrain has a positive effect on the LOS in that direction of travel; this effect can be estimated by an operational analysis procedure.

The HCM analysis procedure provides a methodology for determining the appropriate section length for analysis, the percent time spent following, the average speed, and the LOS, among other metrics. However, the methodology is intended only for the analysis of a single passing lane section and its adjacent upstream and downstream two-lane sections. For analysis of the interactions between two or more passing lane sections (i.e., the support for appropriate passing lane spacing), the HCM recommends using simulation modeling and provides guidance on selected variables to consider in the simulation.

Safety

In data from 22 sites in four states, Harwood and St. John found the crash rate reduction effectiveness of passing lanes to be 9% for all crashes and 17% for fatal and injury crashes (*7*). A later study by Harwood et al. (*8*) used those results, plus findings from Rinde (*9*) and Nettelblad (*10*), to conclude that the crash modification factor for a conventional passing or climbing lane added in one direction of travel on a two-lane highways is 0.75 for total crashes in both directions of travel over the length of the passing lane from the upstream end of the lane addition taper to the downstream end of the lane drop

taper. This crash modification factor assumed that the passing lane is operationally warranted and that the length of the passing lane is appropriate for the operational conditions on the roadway.

Mutabazi et al., in research for the Kansas DOT, evaluated the safety effect of seven passing lane sections using two methods: before–after analysis and cross-sectional analysis (*11*). In the before–after analysis, the frequency for the before period and the frequency for the after period were estimated by the average of observed frequencies for each period. The crash frequency for the after period, assuming that the improvement (i.e., provision of passing lanes) was not implemented, was predicted from the trend of the before period and then adjusted for changes in traffic volume and differences in period lengths between before and after periods. Then, the crash reduction from the improvement was determined with the assumption that the crash frequency of the after period (with improvements) follows a Poisson distribution. From this analysis, they concluded that the data were not sufficient to detect any safety benefit from the improvements. However, in a cross-sectional analysis in which highways with passing lanes were compared with similar highways without passing lanes, the sections with passing lanes had significantly fewer crashes (0.5% significance level) than the average rural two-lane road in the state, on the basis of the number of crashes per million vehicle miles traveled.

FIELD STUDIES

Background

In a recent Texas research project (*2*), Texas DOT districts were surveyed for locations where passing lanes were present. Researchers used responses to identify two Super 2 locations for data collection: SH-121 in the Paris district (northeast Texas) and US-183 in the Yoakum district (southeast Texas). Both two-lane rural highways had posted speed limits of 70 mph (113 km/h); the US-183 corridor allowed passing in the opposing lane between passing lanes, and the SH-121 site did not. Both sites contained several intersections with minor roads within the boundaries of the study corridor.

The field study documented driver behavior and traffic conditions at the beginning and ending of the passing lanes and collected traffic volume, classification, speed, and headway data before, within, and beyond each passing lane. These data were used to calibrate the traffic simulation model, the Traffic Analysis Module (TAM) within the FHWA Interactive Highway Safety Design Model, used later in the research project to develop estimates of passing lane impacts across ranges of traffic volumes found along two-lane roadways in Texas.

Field Data Collection

Two types of data collection equipment were employed for collecting passing lane data.

First, a video trailer with a telescoping mast was used to capture driver behavior approaching each of the four passing lane sections studied. A second trailer was used to record driver merging behavior at the downstream end of each passing lane section. Two cameras atop the mast allowed analysts to observe a field of view that included a short roadway segment preceding the passing lane, the expansion taper, and an additional distance approximately 0.25 mi (0.4 km) downstream of the beginning of each passing lane. At passing lane termini, a single camera was used, and the field of view included the lanes approaching the reduction taper, the taper itself, and a short distance downstream. Digital video recording equipment was used to create a permanent 24-h site-visit video for passing lane beginning and ending points for each of the four passing lanes studied.

Second, portable traffic analyzers, or "plate counters" were used. Sensors determined vehicle count, speed, and classification data, and they recorded speed, classification, and headway data for each vehicle passing over the sensor. Counter data were collected for each location before, within, and after each passing lane.

FIELD DATA ANALYSIS

Passing Lane Entrance

Video data collected at the beginning of the passing lane for each of the four study sites included lane selection and observed passing behavior. Analysts recorded motorists' lane selection by vehicle type and whether vehicles entering the passing (left) lane were initiating a passing maneuver at the upstream end of the passing lane. The data for each site are summarized in Table 2.

Table 2 shows that just over 20% of vehicles on SH-121 entered the left lane, and most of the vehicles entering the left lane were passing vehicles. Nearly every vehicle entering the left lane on northbound SH-121 was a passing vehicle, indicating better compliance with the Texas law requiring use of the left lane for passing only, but both directions show that there is a high level of motorist understanding and compliance with the passing lane. Heavy vehicles composed less than 5% of the traffic stream in both directions, and trucks consistently used the right lane as they entered the passing lane section.

Passing lane operations along southbound US-183 were largely similar to SH-121, but some key differences were noted on northbound US-183: almost 50% of vehicles entering the passing lane section did so in the left lane, but only 25% of those vehicles did so to pass a

TABLE 2 Passing Lane Entrance Data

		Vehicles Entering Left Lane		Trucks	
Site	24-h Vehicle Count	Percent of 24-h Count	Percent Passing	Percent of 24-h Count	Percent Entering Right Lane
SH-121 NB[a]	2,731	22.1	91.9	4.3	92.4
SH-121 SB[a]	2,736	20.8	65.9	4.9	80.9
US-183 NB	2,844	48.1	24.9	7.8	73.6
US-183 SB	2,780	13.2	80.3	10.4	95.3

NOTE: NB = northbound; SB = southbound.
[a]These sites had inadequate lighting to reduce the data collected during nighttime hours, so the vehicle counts shown are lower than the actual 24-h count.

TABLE 3 Passing Lane Departure Data

Site	24-h Vehicle Count	Left-Lane Vehicles		Trucks		Merge Conflict			
		Percent of 24-h Count	Percent Passing	Percent of 24-h Count	Percent in Right Lane	None	Low	Medium	High
SH-121 NB	3,495	27.5	66.2	6.9	66.8	63	37	17	0
SH-121 SB	3,250	28.0	62.5	7.3	72.6	156	97	24	2
US-183 NB	2,664	24.5	41.2	7.1	79.9	186	59	16	0
US-183 SB	2,642	18.0	55.6	7.5	90.9	162	69	27	0

slower vehicle. The percentage of trucks in the northbound direction was just under 8%, and roughly 75% of trucks entered the right lane of the passing lane section.

Passing Lane Terminus

At the end of each passing lane, video data were reduced to determine lane selection by vehicle classification and the presence and severity of merging conflicts between passing and passed vehicles (see Table 3). If analysts observed potential merging conflicts between vehicles at the end of the passing lane, they rated the occurrences as follows:

- None: vehicles with a headway less than roughly 3 s, but no merge conflict;
- Low: slight corrective steering action or speed change (no braking);
- Medium: some braking; and
- High: both braking and swerving to avoid collision.

Driver behavior at the end of each passing lane was more consistent than driver behavior at the beginning of passing lanes for the sites under investigation. Drivers chose the left lane between 18% and 28% of the time, with some increases in left lane usage noted during higher-volume (peak) periods of the day. Passing percentages were slightly lower than those observed for the start of the passing lane and varied between 41% and 66%, suggesting that many left-lane vehicles complete their passing maneuvers early in the passing lane section and do not change lanes before leaving the section. Truck use of the right lane remained high, but was also slightly lower than that observed at the start of the passing lanes. Truck use of the right lane, passing behavior, and the percentage of vehicles using the left lane are all likely influenced by driver reactions to the passing lane terminus.

The rate of merging conflicts observed in the field was consistently low across the study sites. Merging events rated as medium or high occurred with greater frequency during higher-volume peak periods, but on average the number of medium or high merging conflicts was less than one per 100 daily vehicles. The frequency of low merging conflicts, or absence of conflicts, indicates that the merge design provides an area relatively free of merge conflicts for the traffic to return to single-lane operation at the traffic volumes observed.

SIMULATION ANALYSIS

Creating Models

As recommended by the HCM, researchers used simulation to estimate operational effects of passing lanes. The goal of the simulation modeling process was to identify if or when a certain type of passing lane application may be more beneficial for operations. Researchers used detailed roadway data (e.g., horizontal and vertical alignment, cross section) from the US-183 study site to create the base model in TAM for evaluation and to calibrate the model's parameters to be based on traffic conditions observed from the field data.

There were five key study variables: ADT, terrain, proportion of heavy vehicles, passing lane length, and number of passing lanes; the specific values for each variable are listed in Table 4. Researchers completed 648 simulation scenarios; every combination of variables in Table 4 produced 216 scenarios, which were each run three times. Each simulation was conducted to evaluate a hypothetical peak hour (i.e., 60-min duration), and each run used a fixed-length 10-mi (16.1-km) study corridor.

Researchers extracted relevant measures of effectiveness (MOEs) from the simulation output files. The following MOEs were selected for this analysis and calculated for the entire study corridor for each direction of travel and both directions combined:

- Percent time spent following: estimates of the portion of time within the corridor that a driver would like to pass another vehicle but cannot because of limited sight distance or oncoming vehicles; it is the percentage of travel time that vehicles were impeded by other vehicles and not traveling freely, according to the TAM logic, which was approximately equivalent to a 4-s headway when applied to the field data.
- Average total delay: the algebraic average of total delay per vehicle, consisting of two parts, geometric and traffic delays. It is the difference between the measured travel time and the travel time on an ideal (straight and level) roadway alignment with zero traffic impedance.
- Number of passes: the total number of times vehicles overtake other vehicles during the simulation. Valid passing maneuvers can occur on a two-lane section (e.g., passing in the opposing lane) or on a passing lane section.

Modeling Results

Researchers investigated operational performance for specific combinations of variables. Two examples are given in Figures 2 and 3,

TABLE 4 Study Variables for Simulation Modeling

Variable	Values
ADT (vpd)	3,000, 5,333, 7,667, 10,000, 12,333, 14,667
Terrain	Level, rolling
Proportion of heavy vehicles (%)	10, 20
Passing lane length (mi)	1, 2, 3
Number of passing lanes	0, 3, 6

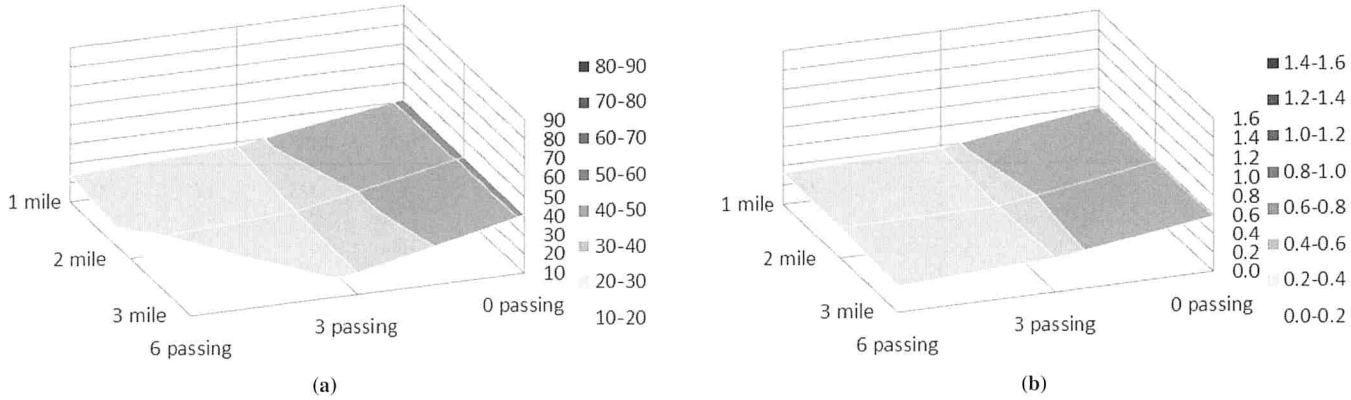

FIGURE 2 Performance measures on passing lane configurations in 10-mi (16.1-km) simulation corridor (3,000 vpd, 10% trucks, level terrain): (*a*) percent time spent following and (*b*) average total delay (minutes per vehicle).

showing three-dimensional contour maps for conditions on the extremes of the study variables. Figure 2 shows the lowest (3,000 vpd) volume level with 10% trucks and level terrain; Figure 3 shows the highest (14,667 vpd) volume level with 20% trucks and rolling terrain. The three-dimensional contour maps in Figure 2 and Figure 3 show the number of passing lanes on the *x*-axis, the length of passing lanes on the *y*-axis, and the MOE on the *z*-axis. The format of the figures uses the degree of slope to reveal the rate of performance improvement. The transition from light shades of gray to dark reveals higher percent time spent following and higher delay, which can also be used as an indicator of performance. Results of these two examples show that although specific values of the MOEs were noticeably different between the ADT levels, adding new passing lanes and extending their lengths helped to improve operational performance in similar ways in both cases.

Analysis of the contour maps for all scenarios indicated the following:

- Improvements from increasing the number of passing lanes were generally greater than improvements from increasing the length. The transition along the *x*-axis (number of passing lanes) in the contour map has a steeper slope and more changes in the color band than along the *y*-axis (length of passing lanes). This finding supports previous research findings that platoons tend to disperse within a certain distance along a passing lane; beyond that distance, little passing occurs. However, an additional lane will help to disperse platoons elsewhere in the corridor.
- Incremental improvements from adding passing lanes along the 10-mi corridor tend to diminish. The improvement from adding three passing lanes to a scenario with no passing lanes is greater than the improvement from adding three more passing lanes to produce a scenario with six passing lanes. From a practical standpoint, as more passing lanes are added, the road will more closely resemble a continuous four-lane corridor, minimizing the cost savings of installing a Super 2 treatment. It is therefore reasonable that incremental operational benefits diminish accordingly.
- Similarly, the difference between MOEs for the scenarios 1 mi (1.6 km) and 2 mi (3.2 km) in length was greater than the difference between MOEs for the scenarios 2 mi (3.2 km) and 3 mi (4.8 km) in length, suggesting a trend of diminishing returns for added passing lane length.

Researchers also evaluated how changing ADT and other variables affects MOEs under a fixed passing lane configuration. Two examples are given in Figures 4 and 5. The former shows results from examining MOEs in a scenario with no passing lanes; the latter displays results for the same performance measures in a scenario with six passing lanes, each 1 mi (1.6 km) in length.

The number of passes shown in Figure 4 is noteworthy in that it does not have a definable mathematical relationship (i.e., linear,

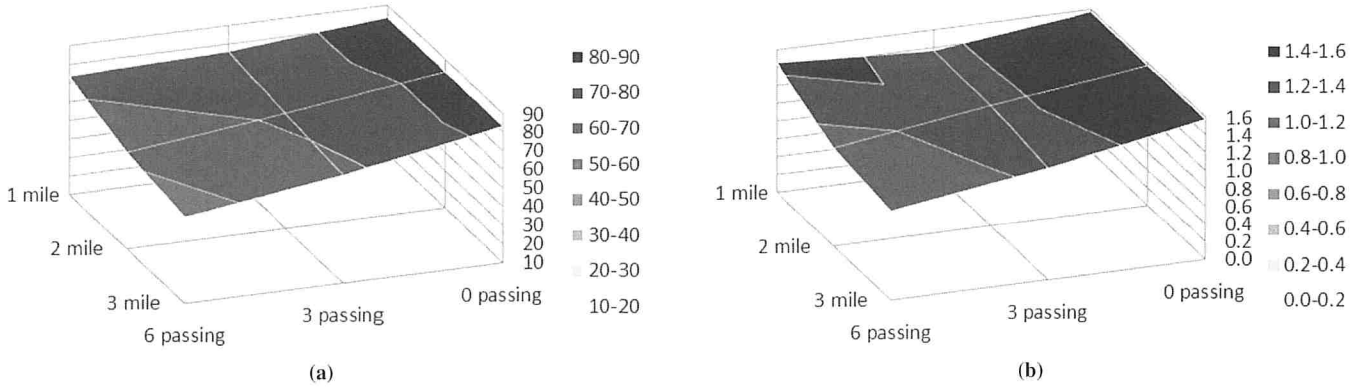

FIGURE 3 Performance measures on passing lane configurations in 10-mi (16.1-km) simulation corridor (14,667 vpd, 20% trucks, rolling terrain): (*a*) percent time spent following and (*b*) average total delay (minutes per vehicle).

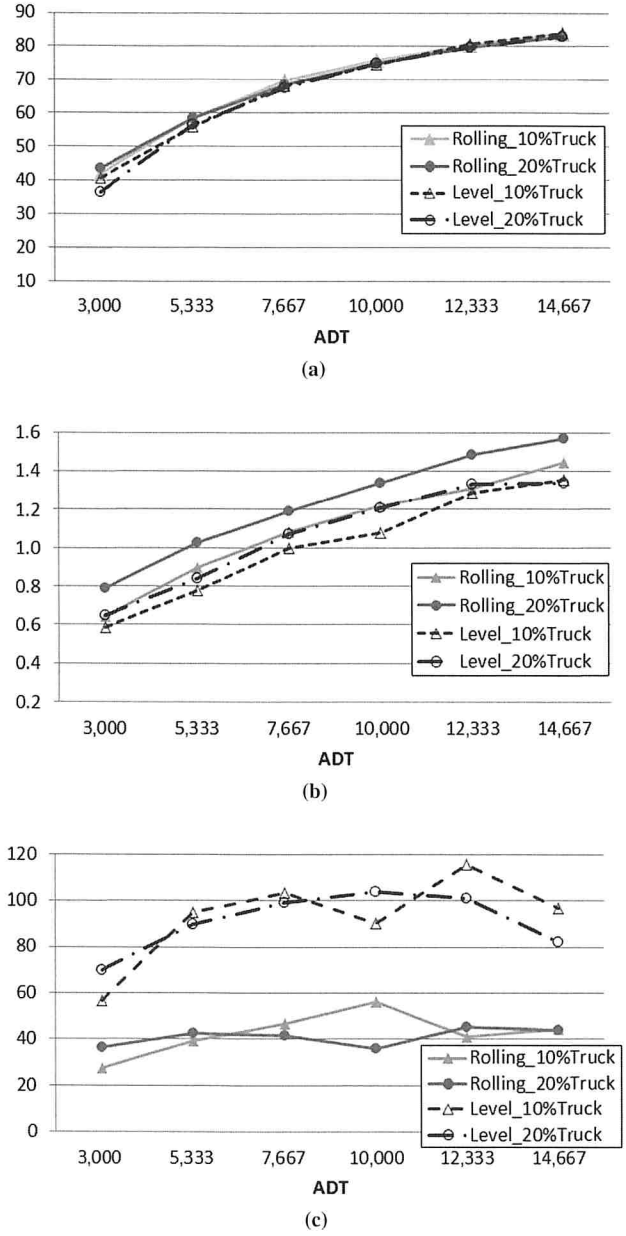

FIGURE 4 Performance measures on varied geometric and traffic conditions [no passing lanes in 10-mi (16.1-km) simulation corridor]: (*a*) percent time spent following, (*b*) average total delay, and (*c*) number of passes.

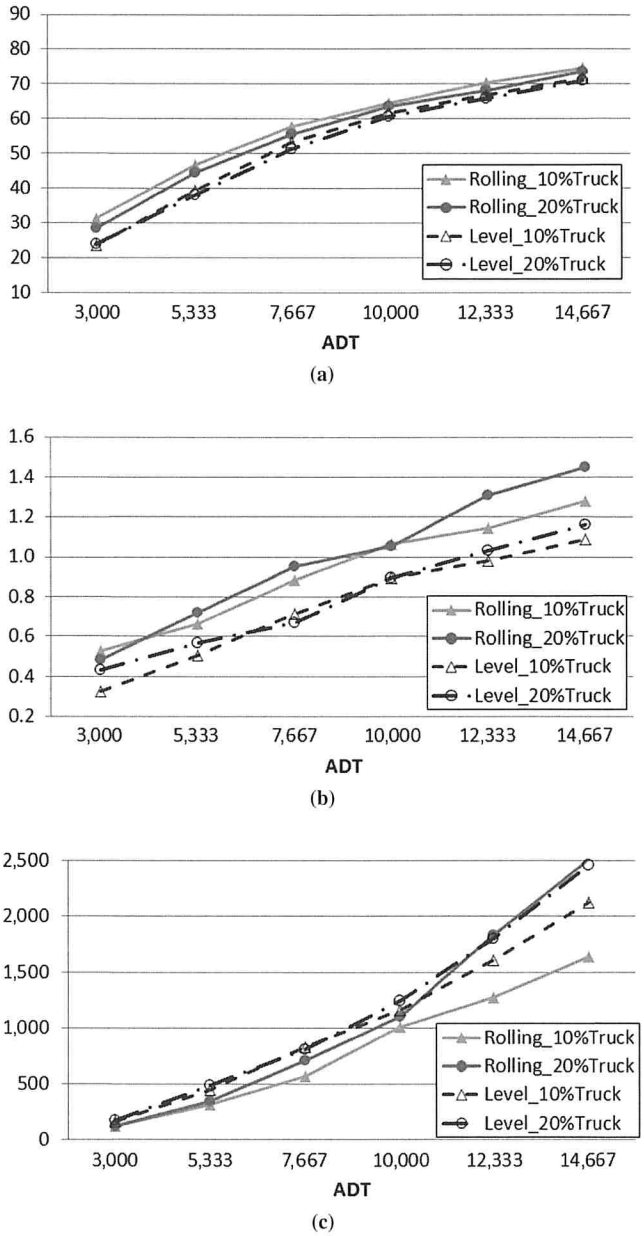

FIGURE 5 Performance measures on varied geometric and traffic conditions [six passing lanes, each 1 mi (1.6 km) in length, in 10-mi (16.1-km) simulation corridor]: (*a*) percent time spent following, (*b*) average total delay, and (*c*) number of passes.

exponential) relative to increases in ADT. Instead, the results indicate that the number of passes reached a maximum threshold between ADT values of 7,667 and 12,333 vpd because of the limited passing opportunity present in the scenario with no passing lanes.

The trend lines in Figure 5*c* diverge as ADT increases. One can conclude that as the volume increases, the number of passes increases because there are more opportunities to pass slower vehicles, especially trucks, when a passing lane is provided. However, for a given roadway corridor, the percent time spent following and the average delay also increase as ADT increases. A comparison of results for level terrain and 20% trucks in Figures 4 and 5, however, indicates a delay savings of about 0.18 min (about 11 s) for each vehicle with the addition of six passing lanes at an ADT of 14,667 vpd, a total delay savings of almost 4.3 h during the 1-h simulation period. Similar savings can be calculated for the other scenarios, as shown in Table 5.

SAFETY

Researchers used the empirical Bayes method in this study to evaluate crash history and safety effects at study sites. Additional information is available in the full research report (*2*) and in a companion paper presented at the TRB annual meeting in 2012 (*12*). The key element in the empirical Bayes method is to predict what would have been the expected frequency of target crashes in the after period for each treated site had the treatment not been applied

TABLE 5 Estimated Delay Savings (h) from Installation of Six 1-mi (1.6-km) Passing Lanes in 10-mi (16.1-km) Simulation Corridor

	Rolling Terrain		Level Terrain	
ADT	10% Trucks	20% Trucks	10% Trucks	20% Trucks
3,000	0.6	1.5	1.3	1.1
5,333	2.0	2.7	2.4	2.4
7,667	2.5	3.1	3.7	5.2
10,000	2.5	4.7	3.1	5.2
12,333	3.4	3.6	6.2	6.2
14,667	3.9	2.9	6.5	4.3

(*13*). The empirical Bayes method was superior to other available methods in that it predicted the expected number of target crashes of a site from two pieces of information: (*a*) actual number of crashes at treated sites during the before period and (*b*) crashes at reference sites with similar geometric characteristics. This prediction was compared with the actual number of crashes after treatment.

Using results from the Texas DOT questionnaire and additional information about other Super 2 sites, researchers identified five corridors for crash data analysis (see Table 6). After determining the correct beginning and ending mile points for all passing lane segments, researchers extracted data from Texas DOT databases to identify crashes within each segment. Two categories of crashes were developed for the evaluation: segment-only crashes (KABC) and segment and intersection crashes (KABC). Segment crashes include driveway and nonintersection crashes, and intersection crashes include intersection and intersection-related crashes. The abbreviation KABC indicates the crash injury severity types: fatal (K), incapacitating injury (A), nonincapacitating injury (B), and minor injury (C), respectively.

Using the empirical Bayes methodology, researchers defined an appropriate reference group of untreated sites and developed a safety performance function for that group. Yearly correction factors were then identified, enabling the calculation of empirical Bayes estimates and variances for the period before construction at each site as well as the predicted number of crashes after construction. Before periods ranged from 5 to 9 years, and after periods were approximately 2 to 5 years; all occurred between 1997 and 2009.

In the estimation of overall changes in crashes shown in Table 7, the empirical Bayes estimates were summed over all corridors and compared with the sum of the actual crashes during the after period (see Table 8). The results indicate that the installation of passing lanes led to a statistically significant crash reduction of 35% for segment-only crashes (KABC) and 42% for segment and intersection crashes (KABC) on the study corridors. These findings were statistically significant above the 95% confidence level, which indicates that the reduction in crashes can be attributed with a high degree of certainty to the Super 2 treatments. The results were consistent with findings of previous safety studies of Super 2 corridors, which showed improvements in safety with installation of passing lanes, even at traffic volumes higher than those considered under previous guidance in Texas.

CONCLUSIONS

From the analysis of field data collected at the Super 2 study sites, simulated Super 2 corridors, and historical crash data at existing Super 2 sites, researchers drew the following conclusions:

- Results agree with previous research that Super 2 corridors improve operations on rural two-lane highways; in addition, the benefits of passing lanes continue to be realized as ADT levels increase to near capacity.
- Observations of lane selection at the entrance to the passing lane section indicate that large numbers of vehicles began passing maneuvers at the beginning of the section; however, not all vehicles in the left lane actually used the left lane for passing, contrary to Texas law.
- The greatest benefits from passing lanes appear to be within about the first mile of their length; this finding is consistent with previous research findings that most passing occurs within that distance.
- Benefits from adding passing lanes diminish as the corridor approaches a continuous four-lane alignment.
- The percent time spent following increases as ADT increases, but at a decreasing rate.
- In most simulation scenarios, the number of passes for 20% truck volume is higher than that for 10% trucks. It is believed that the higher percentage of slower trucks provided more opportunity for overtaking by faster passenger cars, using both the opposing lane and the passing lanes to complete passing maneuvers.
- The largest difference in number of passes by terrain is in the scenario with no passing lanes. For scenarios with three or six passing lanes, regardless of passing lane length or truck percentage, the number of passes is similar for rolling and level terrain.

TABLE 6 Passing Lanes Identified for Safety Study

Texas District	Highway	Control Section	Number of Passing Lane Segments	Passing Lane Length (mi)	Passing Lane Type	2007 AADT[a] (vpd)
Paris (Corridor 1)	SH-121	549-1	4	6.18	Alternating	6,500–6,900
Paris (Corridor 2)	SH-121	549-2	6	7.39	Alternating	6,300–7,700
Wichita Falls	US-283	124-2	10	16.45	Alternating	1,850
Yoakum	US-183	153-2	4	7.08	Separated	5,000
Bryan	SH-30	212-4	4	6.76	Alternating/separated	4,300
Total	—	—	28	43.86		

NOTE: 1 mi = 1.61 km; — = not applicable.
[a] Annual average daily traffic (AADT) amounts were obtained from the Texas DOT statewide planning map, at http://www.dot.state.tx.us/apps/statewide_mapping/StatewidePlanningMap.html.

TABLE 7 Results of Crash Analysis

Location	Segment Crashes						Segment and Intersection Crashes					
	Before			After			Before			After		
	No. of Days	Actual No. of Crashes During Period	Expected No. of Crashes Without Treatment	No. of Days	Actual No. of Crashes During Period	Expected No. of Crashes Without Treatment	No. of Days	Actual No. of Crashes During Period	Expected No. of Crashes Without Treatment	No. of Days	Actual No. of Crashes During Period	Expected No. of Crashes Without Treatment
Paris (Corridor 1), SH-121 (549-1)	1,825	14	14.5	2,147	14	12.9	1,825	20	31.47	2,147	18	29.69
Paris (Corridor 2), SH-121 (549-2)	1,825	25	25.0	2,223	16	23.9	1,825	30	43.11	2,223	18	43.85
Bryan, SH-30 (212-4)	2,564	32	31.2	1,242	4	14.0	2,564	34	32.980	1,242	4	15.896
Yoakum, US-183 (153-2)	3,350	58	57.4	579	5	8.2	3,350	65	75.94	579	5	11.17
Wichita Falls, US-283 (124-2)	2,876	19	19.8	582	1	2.6	2,876	21	38.41	582	1	5.48

• Empirical Bayes analysis of five Super 2 corridors showed that there is a statistically significant crash reduction of 35% for segment-only crashes (KABC) on the study corridors, as compared with the expected number of crashes without passing lanes. There was a similar 42% reduction for segment and intersection crashes (KABC). This finding is consistent with findings of previous safety studies of Super 2 corridors, which show improvements in safety with installation of passing lanes, even at traffic volumes higher than those considered under previous guidance in Texas.

RECOMMENDATIONS FOR SUPER 2 DESIGN

Researchers made the following recommendations for Super 2 design guidance in Texas (*2*):

• Where terrain, available budget, and other considerations allow, providing an additional passing lane in a corridor is preferred over adding length to an existing lane.

• ADT should not be used as the primary criterion for installing passing lanes, although it can be used to prioritize candidate sites, especially when considering truck volumes. In lieu of ADT limits, designers should consider existing right-of-way width, terrain, and structures to evaluate the feasibility of a Super 2 corridor and determine the best locations to install passing lanes with a minimum of right-of-way acquisition, earthwork, and structure widening.

• The location of major traffic generators should be identified as the proposed alignment is planned. It is preferable to avoid locating high-traffic intersections and driveways within the boundaries of a passing lane. Engineering judgment suggests that, when such generators are unavoidable, they should be located near the midpoint of the passing lane to provide maximum separation from the opening and closing tapers.

• Passing lanes should not be located in areas with restrictive geometry (e.g., sharp horizontal curves) or other impediments to traffic flow (e.g., approaches to urbanized areas). Providing passing lanes downstream of these features is beneficial for dispersing platoons, however.

• Where passing lanes are terminated, sufficient sight distance must be provided to avoid conflicts with oncoming traffic or constraints such as guardrails or narrow bridges. Stopping sight distance is recommended.

TABLE 8 Empirical Bayes Results

Variable	Segment Crashes	Segment and Intersection Crashes
Number of crashes	40	46
Expected number of crashes during after period had passing lanes not been installed	61.73	79.29
Variance	7.41	10.91
Estimated index of effectiveness	0.65	0.58
Standard error	0.11	0.09
95% confidence interval (lower limit)	0.439	0.406
95% confidence interval (upper limit)	0.854	0.753
Percent reduction in the number of crashes	35%	42%

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The Operational Effects of Geometrics Committee peer-reviewed this paper.

Development of Left-Turn Lane Warrants for Unsignalized Intersections

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Left-turn movements at intersections, including driveways—especially movements that are made from lanes that are shared with through traffic—cause delays and adversely affect safety. Although some left-turn warrants have been updated, many agencies still use research performed by M. D. Harmelink in the mid-1960s. Most states use procedures that are based on Harmelink's work, but several limitations have been identified. Economic analysis can provide a useful method for combining traffic operations and safety benefits of left-turn lanes to identify situations in which left-turn lanes either are or are not justified economically. This project used a benefit–cost approach to determine when a left-turn lane would be justified. The steps included simulation to determine delay savings from installing a left-turn lane, crash costs and crash reduction savings determined from safety performance functions and crash modification factors available in the *Highway Safety Manual*, and construction costs. Left-turn lane warrants were developed for rural two-lane highways, rural four-lane highways, and urban and suburban roadways. In addition, warrants for bypass lanes were developed for rural two-lane highways.

Left-turn movements at intersections, including driveways—especially movements that are made from lanes that are shared with through traffic—cause delays and adversely affect safety. Left-turn lanes can provide benefits in safety as well as operations. Left-turn lanes can reduce the potential for collisions by providing safer left-turn operations. They can also reduce delay and improve left-turn capacity by removing stopped left-turn vehicles from the main travel lane.

Although updated warrants have been developed in some jurisdictions for when to provide left-turn lanes, many agencies use research from the mid-1960s (*1*). As indicated in recent research (*2–4*), some values used in the methodology are no longer valid or there are limitations with the procedure. Current conditions require a broader assessment of when to provide left-turn accommodations, such as explicit consideration of safety and delay. Because technical warrants are an important element of the decision-making process, revised warrants were developed as part of NCHRP Project 3-91 (*5*).

The purpose of NCHRP Project 3-91 was to develop an objective and clear process for the selection of left-turn accommodations at unsignalized intersections. This paper provides the methodology and recommendations from using a benefit–cost approach for establishing left-turn lane warrants. NCHRP Project 3-91 was to also provide guidance on the design of left-turn lane accommodations. A document called *Design Guide on Left-Turn Accommodations at Unsignalized Intersections* (*6*) was developed to provide information on left-turn lane designs, traffic control treatments, and case study examples.

LITERATURE REVIEW AND INSTALLATION GUIDELINES

A review of the literature was performed using many sources, including research reports, state and federal design manuals, and handbooks. Although many procedures are in use by various organizations to determine the need for left-turn lanes, several are similar or identical. Most states' criteria are based on values from the AASHTO publication *A Policy on Geometric Design of Highways and Streets* (commonly known as the Green Book) (*7*) or values from *NCHRP Report 279* (*8*), both of which are based on work done in the 1960s by M. D. Harmelink.

The oldest research found on evaluating the need for left-turn lanes at unsignalized intersections was that of Harmelink (*1*) in a paper that was published in 1967. His research provided the foundation for many current left-turn guidelines. Harmelink based his work on a queuing model in which arrival and service rates are assumed to follow negative exponential distributions. He stated that the probability of a through vehicle arriving behind a stopped, left-turning vehicle should not exceed 0.02 for 40 mph, 0.015 for 50 mph, and 0.01 for 60 mph. He presented his criteria in the form of graphs, 18 in all. To use his graphs, the advancing volume, opposing volume, operating speed, and left-turn percentage need to be known. Graphs for speeds of 40, 50, and 60 mph are given, as well as left-turn volumes of 5%, 10%, 15%, 20%, 30%, and 40%.

The Green Book (*7*) contains a table for use in determining the need for a left-turn lane on two-lane highways. Similar tables are also present in the 2004 (*9*), 2001 (*10*), 1994 (*11*), 1990 (*12*), and 1984 (*13*) editions of the Green Book. The values in the tables are based on Harmelink's work.

In 1985, TRB published *NCHRP Report 279: Intersection Channelization Design Guide* (*8*). In that report, data from Harmelink's work are used to establish guidelines for determining the need for

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a left-turn lane. The following advice is provided for unsignalized intersections within new construction:

1. Left-turn lanes should be considered at all median crossovers on divided, high-speed highways.
2. Left-turn lanes should be provided at all unstopped (i.e., through) approaches of primary, high-speed, rural highway intersections with other arterials or collectors.
3. Left-turn lanes are recommended at approaches to intersections for which the combination of through, left, and opposing volumes exceeds the warrants shown in a figure.
4. Left-turn lanes on stopped or secondary approaches should be provided on the basis of analysis of the capacity and operations of the unsignalized intersection. Considerations include minimizing delays to right-turning or through vehicles and total approach capacity.

NCHRP Report 279 also provides guidance for reconstruction or rehabilitation. The report states:

> Addition of left-turn lanes at existing intersections should be considered if safety or capacity problems occur, or if land-use changes are expected to produce significant shifts in local traffic patterns (such as increases in left-turn demand). Left-turn lanes can often be added within existing street widths by removing parking, narrowing of lanes, or a combination of the two. (*8*)

The traffic volume guidelines described for new intersections are also appropriate for evaluating the need for left-turn lanes at existing intersections. The following NCHRP Report 279 guidelines were suggested for safety:

- Left-turn lanes should be considered at intersection approaches that experience a significant number of crashes that involve left turns (rear-end, left-turn angle, or same direction sideswipe). Four or more such crashes in 12 months, or six or more in 24 months, is considered appropriate.
- When room for separate left-turn lanes is not available, traffic control alternatives should be investigated. Such alternatives to left-turn lane implementation include split phasing at signalized intersections (i.e., operating each approach individually) or prohibition of left turns.

A summary of the findings from the literature and the review of state manuals can be placed into two major groups, those based on the Harmelink procedure and those based on other procedures. The research team identified eight states with procedures based on a methodology other than Harmelink's. Additional details are provided in the final report for NCHRP Project 3-91 (*5*).

BENEFIT–COST EVALUATIONS

A benefit–cost approach to justify right-turn deceleration lanes was presented by Potts et al. (*14*). The benefits of the right-turn lane were determined for crash reduction (change in number of crashes) and delay reduction (improvements in arterial capacity from removing the slower-moving vehicles from the main traffic stream). The cost of the right-turn lane reflected construction costs. The NCHRP 3-91 procedure presents a similar approach to determine when a left-turn lane would be justified. The steps include

- Identify an economic analysis procedure,
- Use simulation to determine delay savings from installing a left-turn lane,
- Calculate crash costs and crash reduction savings with safety performance functions and crash modification factors from the *Highway Safety Manual* (*15*), and
- Determine construction cost.

ECONOMIC ANALYSIS PROCEDURE

Economic analysis can provide a useful method for combining traffic operations and safety benefits of left-turn lanes to identify situations in which left-turn lanes are and are not justified economically. The following equation shows how to calculate the benefit–cost ratio:

$$\frac{B}{C} = \frac{\text{delay reduction} + \text{safety improvements}}{\text{construction costs}} \tag{1}$$

When the benefit–cost ratio exceeds 1.0, a left-turn lane is considered economically justified because the benefits are greater than the costs.

The following equations provide an overview of the components used in the evaluation of left-turn lanes:

$$\frac{B}{C} = \frac{\left[\left\{\mathrm{DR}\left(V_{\mathrm{TR}}, V_{\mathrm{LT}}, \mathrm{Hr}\right) \times \mathrm{DC}\right\} + \left\{\left\langle N_{\mathrm{spf}}\left(\mathrm{ADT}_{\mathrm{maj}}, \mathrm{ADT}_{\mathrm{min}}\right)\right.\right.}{\left.\left.\left(1 - \mathrm{CMF}_{\mathrm{LT}}\right)\right\rangle \times \mathrm{AC}\right\}\right] \times \mathrm{USPWF}(i,n)} \tag{2}$$

$$\mathrm{USPWF}(i,n) = \frac{(1+i)^n - 1}{i(1+i)^n} \tag{3}$$

where

B/C = benefit–cost ratio;
$\mathrm{DR}(V_{\mathrm{TR}}, V_{\mathrm{LT}}, \mathrm{Hr})$ = delay reduction (vehicle hours/year) from left-turn lane installation as function of V_{TR}, V_{LT}, Hr;
V_{TR} = major road volume during traffic period of interest (vehicles per hour);
V_{LT} = left-turn volume during traffic period of interest (vehicles per hour);
Hr = number of hours in year for traffic period of interest;
DC = user cost savings from delay reduction ($ per vehicle hour);
$N_{\mathrm{spf}}(\mathrm{ADT}_{\mathrm{maj}}, \mathrm{ADT}_{\mathrm{min}})$ = safety performance function to estimate intersection-related predicted average crash frequency for base conditions;
$\mathrm{ADT}_{\mathrm{maj}}$ = average daily traffic (ADT) for major roadway;
$\mathrm{ADT}_{\mathrm{min}}$ = ADT for minor roadway;
$\mathrm{CMF}_{\mathrm{LT}}$ = crash modification factor for installation of left-turn lane;
AC = user cost savings from crash reduction ($ per crash);
$\mathrm{USPWF}(i, n)$ = uniform series present worth factor as function of i and n;
i = minimum attractive rate of return expressed as decimal (i.e., 0.04 for 4% return);
n = number of years; and
CC = estimated construction cost for left-turn lane ($).

Benefits are composed of two major components: benefit in crash reduction and benefit in reduction of travel time delays.

Both of these terms are multiplied by the uniform series present worth factor to convert them from annual benefit amounts to the present value of a time series of annual benefits.

DELAY

Scenarios

Computer simulation was used to evaluate the operational benefits of a left-turn lane on intersection delay at an unsignalized intersection. The purpose of the effort was to assess the delay savings from installing a left-turn lane. The key operational issue related to left-turn lanes at unsignalized intersections or driveways is the operational delay to traffic on the major street. If there is substantial delay to through traffic caused by queued left-turn vehicles, provision of a left-turn lane can reduce that delay.

The comparison identifies the benefits when the left turns at an existing driveway or intersection are provided a left-turn lane. The total average delay for when a left-turn lane is present is subtracted from the total average delay when a left-turn lane is not present. This difference represents the total average delay savings per vehicle at the intersection on the major roadway.

Simulation

To conduct the operational analysis of left-turn lanes, a microsimulation model was used to measure the impact of left-turn vehicles on intersection delay. The following variables were examined:

- Presence of a left-turn lane (yes or no),
- Number of through lanes on the major street (two or four),
- Left-turn lane volume (20, 60, 100, or 140 vehicles per hour),
- Through traffic volume:
  - Two lanes on the major road: 400, 600, or 800 vehicles per hour per approach and
  - Four lanes on the major road: 400, 600, or 800 vehicles per hour per lane, and
- Traffic speed on the major road (30, 40, or 50 mph).

Assumptions included

- Arrival is random.
- The standard deviation for speeds is 5 mph.
- The critical gap for left-turning vehicles is 5 s.
- The default traffic composition is 98% automobiles and 2% trucks.
- The unsignalized intersections and driveways considered have no traffic control requiring a stop or yield by vehicles on the major street.
- The left-turn lane, when present, is long to avoid queue spillbacks from the lane.

The systemwide measure of performance used was average delay per vehicle for the whole network, measured in seconds. To evaluate delay, a series of simulation modeling runs was conducted in which each volume scenario was modeled twice, with and without a left-turn lane. Ten runs were conducted per scenario. The difference between the average total delays with and without a left-turn lane was then calculated, and this value represented the operational benefit of having a left-turn lane at an existing site. Figure 1 illustrates the delay reductions. A review of the data revealed that delay

- Increases as the number of left-turn vehicles increases,
- Increases as the volume on the major road increases,
- Decreases or is relatively constant as the speed limit increases for two-lane highways,
- Increases or is relatively constant as the speed limit increases for four-lane highways, and
- Decreases or is relatively constant as the number of lanes increases (with the exception of an approach volume of 600 or 800 vehicles per hour per lane, a left-turning volume of 140 vehicles per hour, and a 50-mph speed limit).

To facilitate the evaluations, linear regression was used to generate equations to represent the anticipated delay for different combinations of number of lanes, major road volume, left-turn volume, and posted speed. The regression delay reduction equations [coefficients available in the final report (*5*)] permit automated calculations within a spreadsheet. The result is the ability to test more scenarios in the determination of left-turn lane warrants.

Delay for Entire Year

The simulation provides predictions of delay per vehicle in the system. This value needs to be converted to delay at the intersection for the entire year. To perform the conversion, the assumed number of hours and the percent of the ADT represented by each traffic period are needed. Table 1 provides the assumptions used to convert delay of seconds per vehicle into hours of delay for the year at the intersection.

Travel Time Delay Savings

The national congestion constants provided in the *2009 Urban Mobility Report* (*16*) were used to determine the value of person time. The 2007 value of time was adjusted using the consumer price index for 2007 and 2009; the index is available from the U.S. Bureau of Labor Statistics (*17*). The ratio of the 2009 to 2007 consumer price index value is 214.537 divided by 207.342, which is 1.03. The ratio 1.03 multiplied by $15.47 gives a 2009 value of time of $16.01.

The value represents average cost of time per person. To convert to an average cost of time per vehicle, the cost of time per person is multiplied by the vehicle occupancy factor of 1.25 persons per vehicle. The result is an average cost of time per vehicle of $20.01.

CRASHES

Crash Prediction

The predicted average crash frequency for an intersection can be determined from equations in the *Highway Safety Manual* (*15*). These equations, called safety performance functions, are regression models for estimating the predicted average crash frequency of individual roadway segments or intersections for a set of specific base conditions. Each safety performance function in the predictive method was developed with observed crash data for a set of similar sites. The safety performance functions, like all regression models, estimate the value of a dependent variable as a function of a set of independent variables. In the safety performance functions developed for the

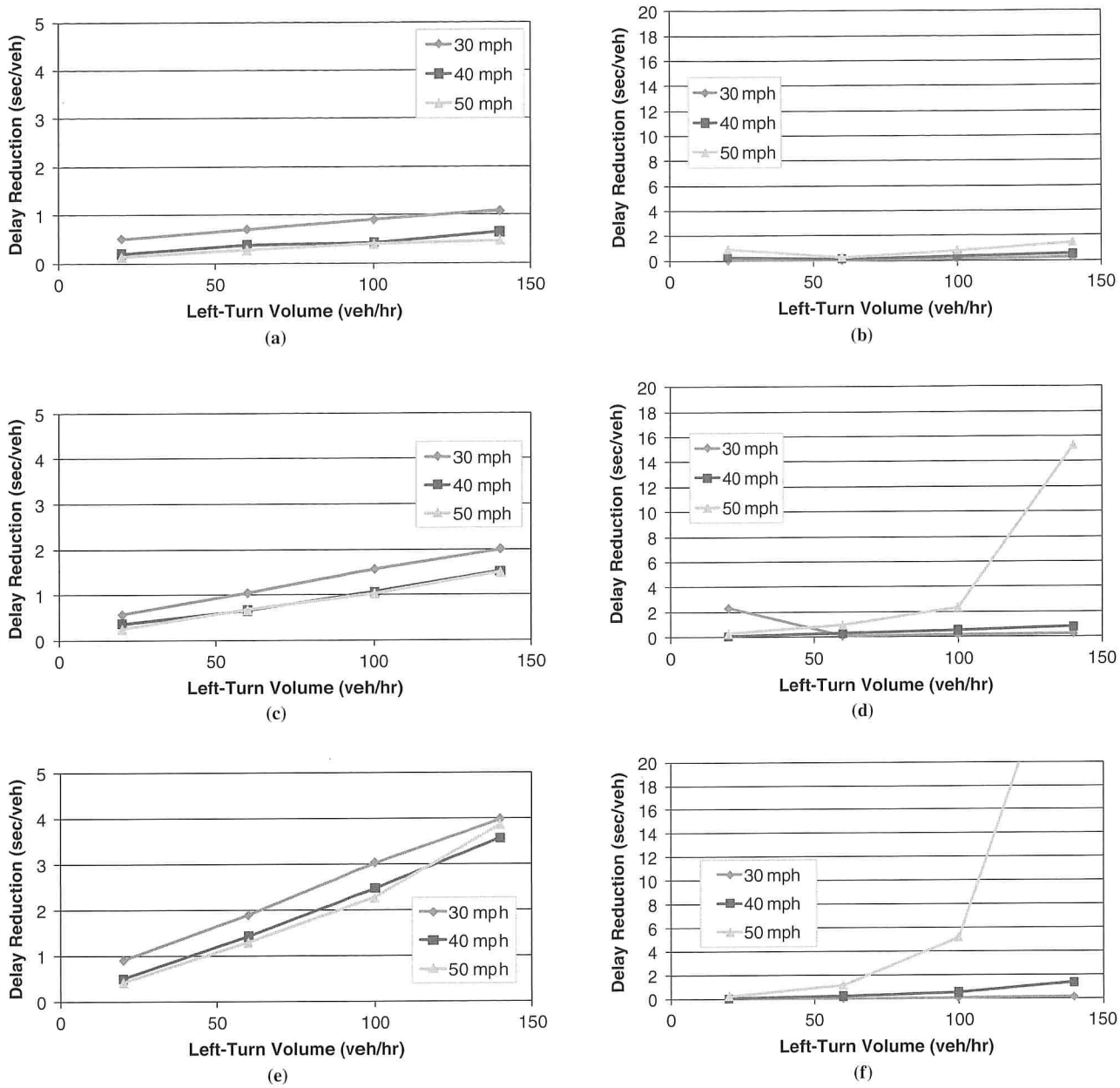

FIGURE 1 Simulation-estimated delay reduction when left-turn lane added at an existing site: (*a*) two lanes, 400 vehicles per hour per approach; (*b*) four lanes, 400 vehicles per hour per lane; (*c*) two lanes, 600 vehicles per hour per approach; (*d*) four lanes, 600 vehicles per hour per lane, (*e*) two lanes, 800 vehicles per hour per approach; and (*f*) four lanes, 800 vehicles per hour per lane.

Highway Safety Manual, the dependent variable estimated is the predicted average crash frequency for a roadway segment or intersection under base conditions, and the independent variables are the annual average daily traffic (AADT) of the roadway segment or intersection legs (and, for roadway segments, the length of the roadway segment).

For rural conditions, different safety performance functions are provided for two-lane and four-lane highways and for three- and four-leg intersections. For urban and suburban arterials, prediction equations are provided for three-leg and four-leg intersections with stop control on the minor road approaches (used in this study) or with signal control (not used in this study). Separate urban and suburban prediction equations are not provided for the number of lanes on the major road approach.

The predicted average crash frequency for base conditions is adjusted with crash modification factors and a calibration factor to adjust for a particular geographical area (not used in this evaluation).

An illustration of the predicted average crashes frequency is shown in Figure 2 for the conditions considered in this evaluation. The graph shows the predicted crashes for a range of major road volumes when the minor road ADT is 2,000 vehicles per day. The predicted number of crashes for intersections on rural four-lane highways and rural two-lane, four-leg intersections is higher than the crash prediction for urban and suburban arterials. The crash prediction in this illustration for rural four-lane, three-leg intersections is similar to urban and suburban three-leg intersections for a given major road ADT.

TABLE 1 Factors Used to Convert Delay from Seconds per Vehicle to Hours per Intersection for a Year

Traffic Period	Number of Hours in Weekday	Number of Hours in Weekend	Hours per Year[a]	Hourly Percentage of ADT During Typical Weekday[b]	Hourly Percentage of ADT During Typical Weekend[c]	Typical Hourly Volume if AADT is 1,000 vpd
Weekday a.m. peak hour	2	0	522	10.0	0	100
Weekday p.m. peak hour	2	0	522	10.0	0	100
Weekday off peak and weekend off peak and peak	5	9	2,243	6.1	6.1	61
Evening	7	7	2,555	2.8	2.8	28
Night	8	8	2,920	1.8	1.8	18
Total	24	24	8,762	na	na	na

NOTE: na = not applicable.
[a]Assume 52.1 weeks/year with 5 days being weekdays and 2 days being weekend days.
[b]Hourly percentage of traffic for given traffic period on a typical weekday.
[c]Hourly percentage of traffic for given traffic period on a typical weekend.

Crash Modification Factor

The crash modification factor for left-turn lanes is available from the *Highway Safety Manual* (*15*). For this evaluation, the assumption was that the intersections had a stop sign on the minor approaches and that only one of the major road approaches would be treated with a left-turn lane.

The crash modification factors for both the rural two-lane and four-lane highway scenarios are 0.56 for a three-leg intersection and 0.72 for a four-leg intersection.

The crash modification factors for the urban and suburban scenarios are 0.67 for a three-leg intersection and 0.73 for a four-leg intersection.

Volumes Used in Crash Prediction for Benefit–Cost Evaluations

Assumptions to determine the traffic volumes used in the evaluation include the following:

- Traffic volumes in both directions of travel on the major road are the same.
- Traffic volume is evenly distributed across the lanes.
- The minor road volume is two times the left-turn volume for three-leg intersections.
- The minor road volume is four times the left-turn volume for four-leg intersections.
- At four-leg intersections, only one major road approach has a left-turn lane. There is no crossing traffic between the minor road legs.

2009 Value of a Statistical Life by Crash Severity

In 2008, a memo was released by the U.S. Department of Transportation about the treatment of the economic value of a statistical life in developmental analyses (*18*). The memo "raises to $5.8 million the value of a statistical life to be used by analysts in the Department of

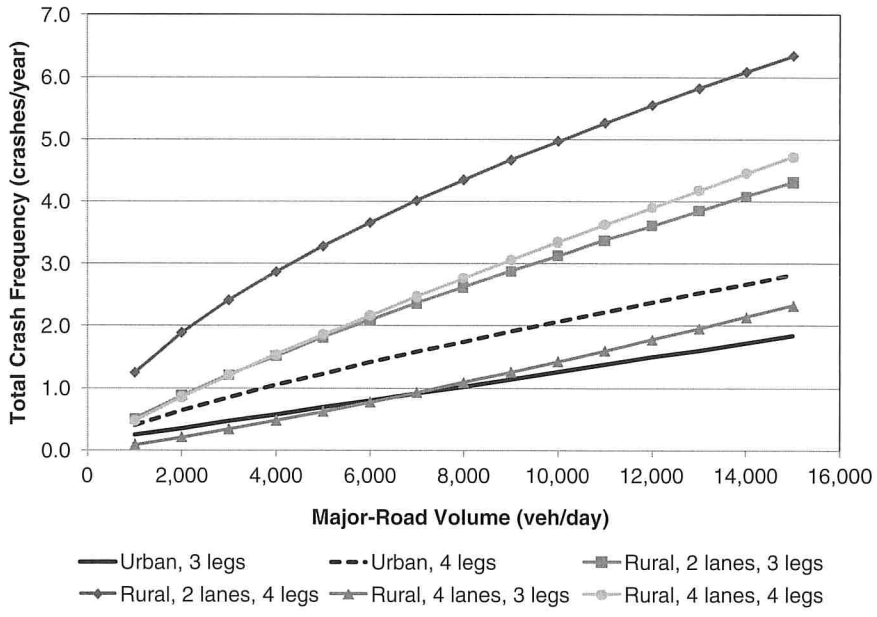

FIGURE 2 Illustration of predicted crash frequency with *Highway Safety Manual* equations.

Transportation when assessing the benefit of preventing fatalities." Researchers used the methodology documented by Council et al. (*19*) and subsequently implemented in the *Highway Safety Manual* to develop an estimate of the value of a statistical life in 2009 dollars by crash severity.

Typical Crash Cost for Three- and Four-Leg Intersections

The cost per crash at a three-leg or a four-leg intersection requires knowing the distribution of crash severity for the different intersection configurations. Table 10-5 in the *Highway Safety Manual* (*15*) provides the default proportions for crash severity levels for rural three-leg and four-leg stop-controlled intersections. The proportions for crash severity for urban and suburban intersections were developed on the basis of information in *Methodology to Predict the Safety Performance of Urban and Suburban Arterials* (*20*).

Also needed is the conversion of the cost per person to the cost per crash. The number of individuals killed or injured in a crash by number of approach legs and rural versus suburban is not readily available. A Texas Department of Transportation study is examining crashes at rural intersections (*21*). Data available for 595 rural intersections provided distributions. For the 1,198 crashes, fatal crashes had 1.09 deaths per crash at the four-leg intersections and 1.46 deaths per crash at the three-leg intersections. Reflecting the multiple conflict points at an intersection, the average number of vehicles involved at a crash ranged between 1.48 and 2.36 vehicles per crash. Table 2 and Table 3 show the calculations to determine typical crash cost using the ranges for comprehensive societal cost for three-leg and four-leg rural highway intersections, respectively.

A study on red-light running at signalized intersections in Texas identified the number of annual crashes and the number of annual injuries by severity level (*22*). From those values, the number of injuries or fatalities was identified. The calculations to determine typical costs for urban suburban arterials are shown in Table 4.

TABLE 2 Typical Crash Cost Calculations for Rural Three-Leg Intersections (2009 U.S. dollars)

Range (Cost)	Crash Severity	Injury Severity	Cost[a, b] ($)	Convert Cost/Person to Cost/Crash[c]	Cost/Crash ($)	Percent of Total Crashes[d] (%)	Extension ($)
Low ($118,000)	Fatality	K	3,234,000	1.46	4,721,640	1.70	82,422
		A	173,500	0.31	35,785		
		B	63,400	1.15	72,910		
		C	36,000	0.00	0		
	A	A	173,500	1.17	202.995	4.00	9,028
		B	63,400	0.29	18,386		
		C	36,000	0.12	4,320		
	B	B	63,400	1.30	82,420	16.60	14,757
		C	36,000	0.18	6,480		
	C	C	36,000	1.22	43,920	19.20	8,433
	PDO	PDO	5,900	1.00[e]	5,900	58.50	3,452
	Total cost/crash					100.00	118,091
Mid ($214,000)	Fatality	K	5,861,700	1.46	8,558,082	1.70	149,393
		A	314,500	0.31	97,495		
		B	115,000	1.15	132,250		
		C	65,200	0.00	0		
	A	A	314,500	1.17	367,965	4.00	16,366
		B	115,000	0.29	33,350		
		C	65,200	0.12	7,824		
	B	B	115,000	1.30	149,500	16.60	26,765
		C	65,200	0.18	11,736		
	C	C	65,200	1.22	79,544	19.20	15,272
	PDO	PDO	10,700	1.00[e]	10,700	58.50	6,260
	Total cost/crash					100.00	214,056
High ($310,000)	Fatality	K	8,489,300	1.46	12,394,378	1.70	216,360
		A	455,500	0.31	141,205		
		B	166,500	1.15	191,475		
		C	94,500	0.00	0		
	A	A	455,500	1.17	532,935	4.00	23,702
		B	166,500	0.29	48,285		
		C	94,500	0.12	11,340		
	B	B	166,500	1.30	216,450	16.60	38,754
		C	94,500	0.18	17,010		
	C	C	94,500	1.22	115,290	19.20	22,136
	PDO	PDO	15,500	1.00[e]	15,500	58.50	9,068
	Total cost/crash					100.00	310,020

NOTE: PDO = property damage-only; K = fatality; A = incapacitating injury; B = nonincapacitating injury; C = minor injury.
[a]Comprehensive societal cost for fatal crash is from Treatment of the Economic Value of a Statistical Life in Departmental Analyses, Memorandum to Secretarial Officers, Modal Administrators, available at http://ostpxweb.dot.gov/policy/reports/080205.htm.
[b]Comprehensive societal cost by crash severity is based on distribution determined using *Highway Safety Manual* data, with costs adjusted to 2009 dollars.
[c]Factors based on 1,189 crashes at 595 rural Texas intersections for the time period of 2003 to 2008 (*5, 20*).
[d]From Table 10-5 of the *Highway Safety Manual* (*14*).
[e]No factor is needed; assumption is that cost reflects cost per crash.

TABLE 3 Typical Crash Cost Calculations for Rural Four-Leg Intersections (2009 U.S. dollars)

Range (Cost)	Crash Severity	Injury Severity	Cost[a, b] ($)	Convert Cost/Person to Cost/Crash[c]	Cost/Crash ($)	Percent of Total Crashes[d] (%)	Extension ($)
Low ($109,000)	Fatality	K	3,234,000	1.09	3,525,060	1.80	66,030
		A	173,500	0.55	95,425		
		B	63,400	0.55	34,870		
		C	36,000	0.36	12,960		
	A	A	173,500	1.40	242,900	4.30	12,392
		B	63,400	0.47	29,798		
		C	36,000	0.43	15,480		
	B	B	63,400	1.42	90,028	16.20	17,384
		C	36,000	0.48	17,280		
	C	C	36,000	1.34	48,240	20.80	10,034
	PDO	PDO	5,900	1.00[e]	5,900	56.90	3,357
	Total cost/crash					100.00	109,196
Mid ($198,000)	Fatality	K	5,861,700	1.09	6,389,253	1.80	119,681
		A	314,500	0.55	172,975		
		B	115,000	0.55	63,250		
		C	65,200	0.36	23,472		
	A	A	314,500	1.40	440,300	4.30	22,463
		B	115,000	0.47	54,050		
		C	65,200	0.43	28,036		
	B	B	115,000	1.42	163,300	16.20	31,525
		C	65,200	0.48	31,296		
	C	C	65,200	1.34	87,368	20.80	18,173
	PDO	PDO	10,700	1.00[e]	10,700	56.90	6,088
	Total cost/crash					100.00	197,929
High ($287,000)	Fatality	K	8,489,300	1.09	9,253,337	1.80	173,330
		A	455,500	0.55	250,525		
		B	166,500	0.55	91,575		
		C	94,500	0.36	34,020		
	A	A	455,500	1.40	637,700	4.30	32,522
		B	166,500	0.47	78,255		
		C	94,500	0.43	40,635		
	B	B	166,500	1.42	236,420	16.20	45,650
		C	94,500	0.48	45,360		
	C	C	94,500	1.34	126,630	20.80	26,339
	PDO	PDO	15,500	1.00[e]	15,500	56.90	8,820
	Total cost/crash					100.00	286,661

[a]Comprehensive societal cost for fatal crash is from Treatment of the Economic Value of a Statistical Life in Departmental Analyses, Memorandum to Secretarial Officers, Modal Administrators, available at http://ostpxweb.dot.gov/policy/reports/080205.htm.
[b]Comprehensive societal cost by crash severity is based on distribution determined using *Highway Safety Manual* data, with costs adjusted to 2009 dollars.
[c]Factors based on 1,189 crashes at 595 rural Texas intersections for the time period of 2003 to 2008 (*5, 20*).
[d]From Table 10-5 of the *Highway Safety Manual* (*14*).
[e]No factor is needed; assumption is that cost reflects cost per crash.

CONSTRUCTION COSTS

State department of transportation websites were searched for information on recently let (2009 or 2010) projects that involved installation of a left-turn lane. Data from the following four states were obtained: Texas (*23*), Louisiana (*24*), Ohio (*25*), and Florida (*26*). From the data, a reasonable range for the cost of constructing a left-turn lane appears to be $100,000 (minimum) to $375,000 (maximum), with an average value of $250,000 (moderate).

EXAMPLE OF CALCULATIONS FOR ADDING LEFT-TURN LANE ON RURAL TWO-LANE HIGHWAY

Annual Delay Savings

The following illustrates the calculations to determine whether a left-turn lane should be considered at a rural two-lane highway site. Assume that the conditions at the site include the following:

- Two lanes,
- Rural location,
- Three legs,
- 50 mph posted speed limit,
- 450 vehicles per hour per lane in the peak hour (10,000 ADT on the major road),
- 100 vehicles per hour turning left in the peak hour, and
- 2,000 ADT on the minor road.

The amount of expected delay reduction that would be attributed to the left-turn lane during the peak hour is calculated as being 0.701 s per vehicle. This value was determined using the regression coefficients available in the final report (*5*); however, it could also be approximated from Figure 1, *a* and *c*. The delay reduction represents the per vehicle delay savings. To determine the savings for all the vehicles at the intersection for that hour, the delay reduction value needs to be multiplied by the number of vehicles at the intersection for that hour, in this example 1,000 vehicles (10% of the 10,000 ADT). Therefore, 701 s (about 12 min) of delay was reduced during the morning peak hour. With 522 h in that traffic period for a year (see Table 1), the

TABLE 4 Typical Crash Cost Calculations for Urban and Suburban Intersections (2009 U.S. dollars)

Range (Cost)	Crash Severity	Cost[a, b] ($)	Convert Cost/ Person to Cost/ Crash[c]	Cost/Crash ($)	Percent of Total Crashes for Three Legs[d] (%)	Extension for Three Legs ($)	Percent of Total Crashes for Four Legs[d] (%)	Extension for Four Legs ($)
Low	Fatality	3,234,000	1.10	3,557,400	1.47	52,461	1.60	57,050
	A	173,500	1.42	246,370	3.47	8,549	3.83	9,439
	B	63,400	1.64	103,976	14.40	14,973	14.43	15,007
	C	36,000	2.10	75,600	16.66	12,591	18.53	14,010
	PDO	5,900	1.00[e]	5,900	64.00	3,776	61.60	3,634
	Total cost/crash					92,350		99,141
	Rounded total (cost/crash)					92,000		99,000
Mid	Fatality	5,861,700	1.10	6,447,870	1.47	95,087	1.60	103,405
	A	314,500	1.42	446,590	3.47	15,496	3.83	17,109
	B	115,000	1.64	188,600	14.40	27,158	14.43	27,221
	C	65,200	2.10	136,920	16.66	22,805	18.53	25,374
	PDO	10,700	1.00[e]	10,700	64.00	6,848	61.60	6,591
	Total cost/crash	167,394		179,701				
	Rounded total (cost/crash)					167,000		180,000
High	Fatality	8,489,300	1.10	9,338,230	1.47	137,711	1.60	149,758
	A	455,500	1.42	646,810	3.47	22,444	3.83	24,780
	B	166,500	1.64	273,060	14.40	39,321	14.43	39,412
	C	94,500	2.10	198,450	16.66	33,053	18.53	36,776
	PDO	15,500	1.00[e]	15,500	64.00	9,920	61.60	9,548
	Total cost/crash	—	—	—	—	242,448	—	260,274
	Rounded total cost/crash	—	—	—	—	242,000	—	260,000

NOTE: — = not applicable.
[a]Comprehensive societal cost for fatal crash is from Treatment of the Economic Value of a Statistical Life in Departmental Analyses, Memorandum to Secretarial Officers, Modal Administrators, available at http://ostpxweb.dot.gov/policy/reports/080205.htm.
[b]Comprehensive societal cost by crash severity is based on distribution determined using *Highway Safety Manual* data with costs adjusted to 2009 dollars.
[c]Factors based on number of injuries or deaths per crash for red-light-running crashes (*5, 21*).
[d]Table 52 from *Methodology to Predict the Safety Performance of Urban and Suburban Arterials* (*19*) shows PDO crashes to be 64.0% for multiple-vehicle crashes at three-leg stop-controlled intersections and 61.6% for four-leg stop-controlled intersections. The remaining 36.0% for three-leg and 38.4% for four-leg intersections were distributed between fatality, A, B, and C using similar proportions as assumed for rural highways.
[e]No factor is needed; assumption is that cost reflects cost per crash.

amount of delay savings for the year is 102 h. Table 5 lists the delay reduction for each traffic period. For this site, the annual delay saving is $4,214, assuming a per hour vehicle cost of $20.01.

Annual Crash Savings

The determination of the savings attributed to crashes begins with calculating the predicted number of crashes for the intersection. The equation for a three-leg, stop-controlled intersection is

$$N_{spf\,2ln,3st} = \exp\left[-9.86 + 0.79 \times \ln\left(AADT_{maj}\right) + 0.49 \times \ln\left(AADT_{min}\right)\right]$$

$$N_{spf\,2ln,3st} = \exp\left[-9.86 + 0.79 \times \ln\left(10{,}000\right) + 0.49 \times \ln\left(2000\right)\right]$$

$$N_{spf\,2ln,3st} = 3.13 \text{ crashes/year} \quad (4)$$

where

$N_{spf\,2\,ln,\,3st}$ = safety performance function from the *Highway Safety Manual* (*15*) to estimate the intersection-related predicted average crash frequency for a three-leg, stop-controlled intersection on a rural two-lane highway;
$AADT_{maj}$ = AADT for major roadway; and
$AADT_{min}$ = AADT for minor roadway.

TABLE 5 Example Calculation of Annual Delay Savings for Rural Two-Lane Highway

Traffic Period	Hourly Factor	Hours/Year	V_{TR} (veh/h)	V_{LT} (veh/h)	DR (s/veh)	DR (s) in Hour	DR (s/year)	DR (h/year)
a.m. peak	10	522	450	100	0.701	701.1	36,5987	101.66
p.m. peak	10	522	450	100	0.701	701.1	36,5987	101.66
Off peak/weekend peak	6.1	2,243	274.5	61	0.010	6.1	13,682	3.80
Evening	2.8	2,555	126	28	0.010	2.8	7,154	1.99
Night	1.8	2,920	81	18	0.010	1.8	5,256	1.46
Total	—	—	—	—	—	—	—	210.57
Dollars per vehicle hour cost	—	—	—	—	—	—	—	$20.01
Annual delay savings	—	—	—	—	—	—	—	$4,213.59

NOTE: V_{TR} = through and right-turn volume on major approach; V_{LT} = left-turn volume; DR = delay reduction; — = not applicable.

The crash modification factor from the *Highway Safety Manual* (*15*) for adding a left-turn lane on one approach for a three-leg intersection is 0.56. The reduction in number of crashes at the intersection can be determined as follows:

$$N_{\text{spf 2ln, 3st}}\left(\text{CMF}_{\text{LT}}\right) = 3.13(0.56) = 1.75 \text{ crashes/year} \quad (5)$$

The expected crash reduction with the addition of the left-turn lane is given by

$$N_{\text{spf 2ln, 3st}} - N_{\text{spf 2ln, 3st}}\left(\text{CMF}_{\text{LT}}\right) = 3.13 - 1.75 = 1.38 \text{ crashes/year} \quad (6)$$

The costs per crash in 2009 dollars at three-leg intersections (see Table 2) are

- $118,000 (low range),
- $214,000 (midrange), and
- $310,000 (high range).

Multiplying the cost per crash by the number of crashes that would not occur because of the presence of the left-turn lane results in the following annual crash savings (rounding differences may be present because of use of a spreadsheet to facilitate calculations):

- $162,441 (low range),
- $294,596 (midrange), and
- $426,752 (high range).

Present Worth of Annual Savings and Benefit–Cost Ratio

The delay and crash savings above represent the expected value per year. Assuming a 20-year design life and a 4% return, the annual savings need to be converted to present worth so that the expected construction cost of the left-turn lane can be considered. Following are the calculations when the construction cost was estimated to be $250,000 and midrange societal was used:

$$\text{USPWF}(i, n) = \frac{(1+i)^n - 1}{i(1+i)^n} = \frac{(1+0.04)^{20} - 1}{0.04 \times (1+0.04)^{20}} = 13.59 \quad (7)$$

$$\frac{B}{C} = \frac{13.59 \times (\$4,214 + \$294,596)}{\$250,000} = 16.2 \quad (8)$$

The benefit–cost ratio of 16.2 indicates that the installation of a left-turn lane is warranted for these conditions.

DEVELOP PRELIMINARY WARRANTS FOR LEFT-TURN LANE

The benefit–cost methodology was applied to a range of major road ADT (1,000 to 15,000) and minor road ADT (200 to 3,000) along with a range of posted speed limits (30 to 60 mph). A service life of 20 years and a minimum rate of return of 4% were assumed. Initially, the minimum ADT for a major road was determined for each posted speed limit; however, minimal differences were identified, primarily because crash costs dominate the calculations, and crash prediction is not a function of posted speed limit. Crash predictions are different for rural and urban conditions; therefore, separate results are presented for the two conditions. Crash predictions also vary by the number of legs at the intersection; therefore, unique results are also presented for three-leg and four-leg intersections.

The FHWA memo on the economic value of a statistical life recommends that a range be considered in an analysis. Therefore, researchers used the following values in the analysis:

- Low range = $3.2 million,
- Midrange = $5.8 million, and
- High range = $8.4 million.

The cost to install a left-turn lane can vary from a minimal amount, such as the cost to restripe a roadway section, to a large value that would occur if right-of-way needs to be purchased in addition to the construction costs. The range of construction costs selected for the analysis is as follows:

- $100,000 (minimum),
- $250,000 (moderate), and
- $375,000 (maximum).

Another suggested approach to account for differences in costs is to use a benefit–cost ratio that is higher than 1.0. The minimum major road ADT for a given left-turn lane volume that gave a benefit–cost ratio of 1.0 and 2.0 was identified in the preliminary review.

Researchers used all of the ranges of variables described above to develop a series of preliminary warrants for installing left-turn lanes. Observations from a review of those warrants include the following:

- The warrants were much more sensitive to changes when the crash costs were varied than when the construction costs were varied.
- The warrants for two-lane highways are noticeably smaller than the existing Green Book warrants.
- Left-turn lanes are always warranted on four-leg intersections at lower volumes as compared with three-leg intersections.
- Left-turn lanes are warranted on rural two-lane highways at low volumes. As few as five left-turning vehicles crossing 50 vehicles per hour per lane results in benefits that outweigh costs when using midrange or high-range crash costs. Even when using low-range crash costs, the benefits are greater than the costs when the five left-turning vehicles are crossing as few as 150 vehicles per hour per lane for four-leg intersections or 200 vehicles per hour per lane for three-leg intersections.
- Three-leg intersections on urban and suburban arterials have the greatest range in warrant values. Warrants based on midrange crash costs are less than the current Green Book warrants. If a low crash cost is assumed, some of the resulting warrants are higher than some of the Green Book warrants.

The primary observation from the reviews that compare the ranges in value for statistical life and construction cost is that the variations in assumed crash cost have a greater impact on the resulting warrants than the variations in assumed construction cost.

LEFT-TURN LANE WARRANTS BASED ON BENEFIT–COST RATIO

The research team recommends that the midrange crash cost and the moderate construction cost identified as part of this research be considered in developing the final left-turn lane warrant recommendations. Because of the availability of a bypass lane design for rural two-lane

highways that is easier to construct than a left-turn lane, different benefit–cost ratios were considered for rural two-lane highways. On the basis of engineering judgment, the following benefit–cost ratios were selected for the different roadway conditions:

- Use benefit–cost ratio ≥ 1.0 for urban and suburban arterials,
- Use benefit–cost ratio ≥ 1.0 to benefit–cost ratio < 2.0 for rural two-lane highways to warrant a bypass lane,
- Use benefit–cost ratio ≥ 2.0 for rural two-lane highways to warrant a left-turn lane, and
- Use benefit–cost ratio ≥ 1.0 for rural four-lane highways to warrant a left-turn lane.

The suggested warrants are shown in Figure 3. For rural two-lane highways, a left-turn lane is warranted for as low as five left-turning vehicles in 1 h when opposed by 50 major road vehicles per hour. For rural four-lane highways, a left-turn lane is warranted when five left-turn vehicles oppose 50 vehicles per hour per lane for a four-leg intersection or 75 vehicles per hour per lane for a three-leg intersection. For urban four-leg intersections, a left-turn lane is to be considered for one vehicle turning across 50 vehicles per hour per lane in the peak hour. Urban three-leg intersections had the biggest range of warrant recommendations and the highest volumes, from 50 vehicles turning across 100 vehicles per hour per lane in the peak hour to 5 vehicles opposing 450 vehicles per hour per lane.

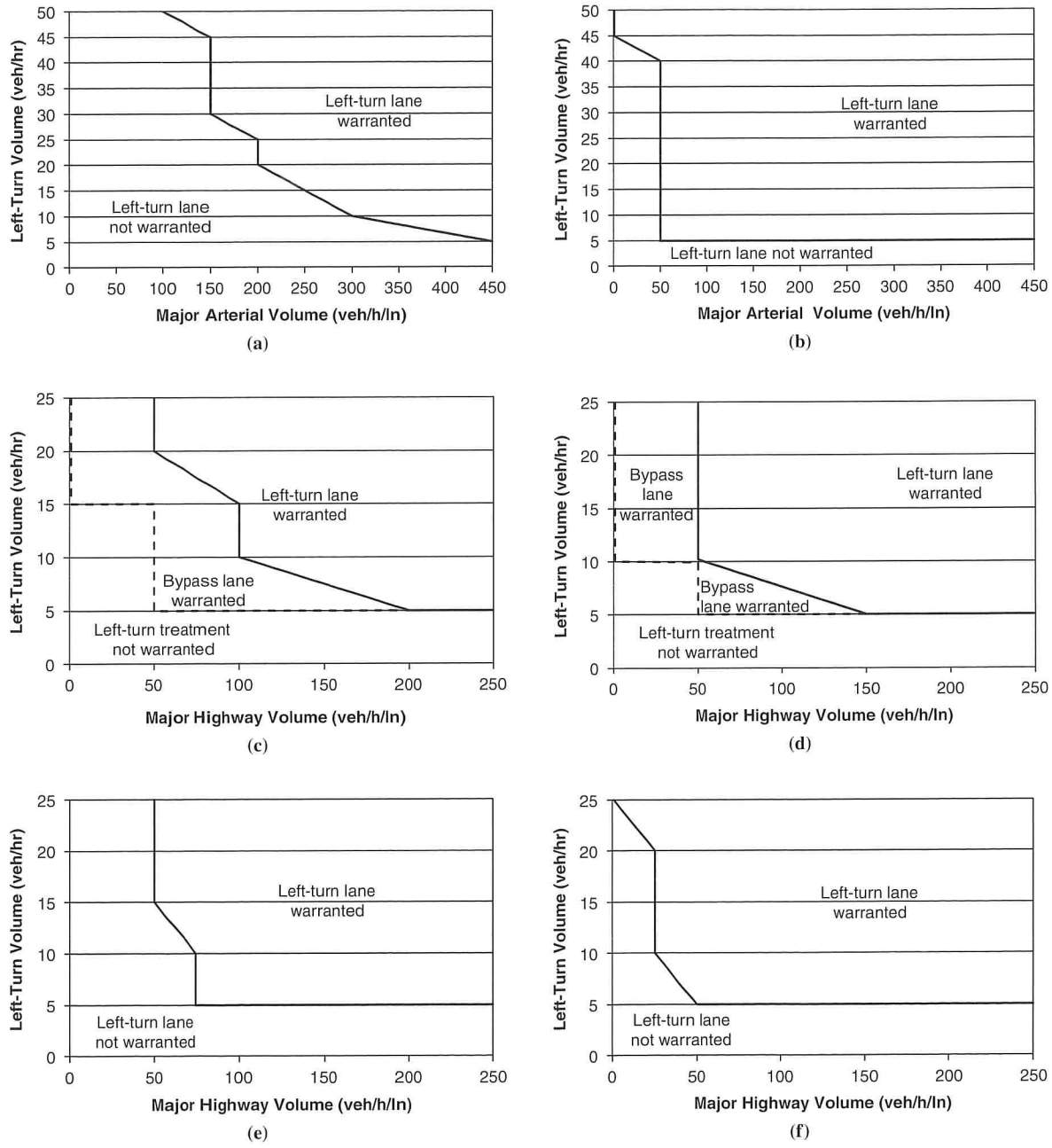

FIGURE 3 Suggested left-turn lane warrants from results of benefit–cost evaluations: (*a*) urban and suburban, three legs; (*b*) urban and suburban, four legs; (*c*) rural two-lane highways, three legs; (*d*) rural two-lane highways, four legs; (*e*) rural four-lane highways, three legs; and (*f*) rural four-lane highways, four legs.

CONCLUSIONS

Left-turn movements at intersections, including driveways—especially movements that are made from lanes that are shared with through traffic—cause delays and adversely affect safety. The goals for a recent NCHRP project were to develop an objective and clear process for the installation of left-turn lanes at unsignalized intersections and provide guidance on the design of these lanes.

Left-turn lanes can reduce the potential for collisions and improve capacity by removing stopped vehicles from the main travel lane. Left-turn lane warrants were developed as part of NCHRP Project 3-91, with an economic analysis procedure for rural two-lane highways, rural four-lane highways, and urban and suburban roadways. NCHRP Project 3-91 also developed the *Design Guide on Left-Turn Accommodations at Unsignalized Intersections* (*6*), which discusses left-turn lane designs, traffic control treatments, and case study examples.

The conclusions and recommendations developed from this research are

- Left-turn lane warrants (see Figure 3) were developed using an economic analysis procedure for
  - Rural two-lane highways,
  - Rural four-lane highways, and
  - Urban and suburban roadways.
- The methodology presented could also be used if a transportation agency has available local values for delay reductions resulting from the installation of a left-turn lane, crash frequency or crash predictions, crash reduction factors, crash costs, and construction costs.

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Safety and Economic Impacts of Converting Two-Way Frontage Roads to One-Way Operation

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In some states (e.g., Texas), frontage roads have been a design solution for providing access along rural freeways and access-controlled principal arterials. In rural and lesser-developed urban areas, the frontage roads are usually operated as two-way facilities because of relatively long distances between interchanges. As areas become more urban and the adjacent land is developed, traffic volumes increase, and as interchange spacing decreases, it becomes desirable to convert the frontage roads to one-way operation. There is a need to objectify safety impacts of frontage road conversion, and business and property owners are often concerned with economic impacts related to access, business activity, and property values. Recognizing these needs and concerns, the Texas Department of Transportation contracted with the Texas A&M Transportation Institute to investigate the safety and economic impacts of converting two-way frontage roads to one way. Researchers investigated eight sites throughout Texas: both conversion sites and comparison sites that remained two way. Researchers developed 12 crash modification factors by crash severity and crash type for frontage road conversion. A sample application is provided in this paper. To assess economic impacts, researchers obtained parcel-level appraisal data and found overall increases in appraised values. Researchers surveyed business owners and managers and customers. Researchers found that business owners and managers are typically concerned with access, gross sales, and ramp and interchange locations and spacing. Finally, researchers identified lessons learned with the crash data for those performing safety analyses.

Frontage roads provide access to adjacent properties and maintain circulation on each side of the highway (*1*). They usually run parallel to the main traveled way and are typically on both sides of the roadway. In rural and lesser-developed urban areas, frontage roads are usually operated as two-way facilities because of relatively long distances between interchanges. As areas with two-way frontage roads become more urban and adjacent land is developed, traffic volumes increase and safety and operational issues become concerns. When this development occurs, consideration is often given to converting the frontage road from two way to one way.

One challenge is to effectively communicate to stakeholders and interested parties the benefits provided by converting frontage roads and what types of impacts may reasonably be expected. Historically, frontage road conversions have been made with the objective of improving traffic operations and safety. Operations are improved by removing conflicts at entrance and exit ramps where opposing traffic must yield to traffic entering and exiting the main lanes. Cross-street signalization is also typically more efficient. When conversions are implemented, conflicts for traffic entering and exiting the main lanes are reduced (i.e., no crossing conflicts remain).

RESEARCH OBJECTIVES

Because of the need for objective information about the safety and economic impacts of frontage road conversion, the Texas Department of Transportation (DOT) funded research with the following objectives (*2*):

1. Develop crash modification factors (CMFs) that can be used to communicate the types of safety impacts that have been experienced and can be expected,
2. Develop accurate information that can be used to communicate the types of economic impacts that have been experienced and can be expected, and
3. Identify any issues in crash data obtained as a result of this analysis for future studies.

BACKGROUND

Conversion Safety Impacts and Warrants

Published research on frontage road conversions from two way to one way is relatively limited. The earliest research on frontage roads was performed in Texas by Woods et al. beginning in the early 1980s. In one of these studies, researchers identified operational and safety problems associated with two-way frontage roads (*3*). Crash and operational data were collected at 22 study sites, and crash frequency and rate on frontage road segments and ramps were analyzed. Researchers reported that slip ramps had the most frequent erratic driving maneuvers, and the average daily traffic and level of area development had a significant effect on crashes at intersections of ramps and frontage roads.

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In the follow-up study, Woods et al. evaluated crash frequency and severity after two-way frontage road operations were converted to one way (*4*). Nine frontage roads with segment lengths between 0.9 and 9.5 mi were selected for evaluation. Researchers collected data for a minimum of 2 years for each site for those that were converted between 1975 and 1981. Using the Wilcoxon matched pairs signed rank test for their analysis, the authors found a reduction of about 20% in crash frequency after conversion. They did not find any statistically significant reduction in crash severity.

This before-and-after analysis used short periods of before-and-after data (2 years at most for each period), and the study did not include frontage road comparison sites. Such an analysis fails to distinguish the effect of treatment (conversion) from the effects of other factors that might have also changed from the before period to the after period (*5*). As a result, the validity of the conclusion that there is a significant reduction (of about 20%) in crashes after the conversion to one-way operation seems questionable. Transportation safety analysts no longer consider this method reliable (*5*).

In 1984, Woods summarized the results of a series of Texas A&M Transportation Institute (TTI) reports regarding the safety and efficiency of frontage roads and proposed warrants for converting two-way frontage roads to one way (*6*). He examined the data collected in the previous studies. From the data, he reported that the factors affecting crash rates were not the type of ramps but the roadside development and the volume of traffic traveling on the frontage road. Consequently, he proposed two warrants, depending on frontage road volumes and crash experience, on the basis of the crash analysis and erratic driving maneuver for two-way frontage roads. The primary limitation of the analysis is that comparison sites were not included in the analysis to account for site selection and regression-to-the-mean biases.

Messer et al. developed numerical operational warrants for converting two-way frontage roads to one way and applied these warrants to five case studies in Texas (*7*). Researchers established these warrants to provide the most efficient possible traffic flow on a one-way or two-way frontage road. They categorized the warrants in three levels according to different traffic volume characteristics.

A recent study by Gattis et al. examined characteristics before and after frontage road conversion along an approximately 16-mi segment of IH-30 between Little Rock and Benton, Arkansas (*8, 9*). Researchers investigated several characteristics, including the number of crashes, traffic volumes, land use, and business owner opinions. Researchers found that main lane crashes decreased 40% (by 271 crashes), frontage road crashes decreased 30% (by 235 crashes), and ramp crashes increased 20% (by six crashes, from 36 to 42) after the conversion. Researchers indicated that the increase in ramp crashes was relatively insignificant and there was an overall crash reduction of one-third despite a 25% increase in traffic volume. For the crash analysis, researchers did not have comparison sites in the analysis. Researchers noted that 20% to 40% of crash reports were located incorrectly in the summary database. Researchers conducted a detailed review of individual crash reports to correct this error.

Opinion Surveys and Economic Impacts of Frontage Road Conversion

Gattis et al. also surveyed business owners and managers, once within 3 months of the conversion and again 1 year after the conversion (*8, 9*). During the second survey, construction was still under way along the frontage roads even though they had been converted. Researchers hypothesized that the full benefits of the conversion had not been fully realized. Researchers suggested that the most frequently given comment by those interviewed was, "One-way frontage roads are very inconvenient." Researchers attributed this response to the frontage roads being operated as two way for more than 40 years before the conversion. Business owners generally had a negative tone in the survey 1 year after conversion. Researchers also noted that comments taken from the same respondents 1 year after the conversion indicated that some of the business owners were more frustrated with the main lane and interchange construction activity than the conversion of the frontage roads. These responses suggest that the construction activity may have a bigger effect on business than the conversion.

Gattis et al. indicated that it is highly unlikely that no businesses were adversely affected by the conversion to one-way operation (*8, 9*). However, they indicated that in the land use survey, in spite of the displeasure of business owners, any real negative effects were not great enough to cause a change in land use that differed from that experienced by two unchanged routes parallel to the freeway corridor.

In 1987, Stover et al. evaluated the attitude of drivers toward the conversion of two-way frontage roads to one-way frontage roads (*10*). The interviewees included city staff members, council members, real estate appraisers, those with real estate and development interests, and owners of abutting businesses. According to the analysis of survey responses, city staff and council members were strongly in favor of one-way frontage roads. However, business owners, real estate representatives, and developers favored two-way frontage roads because they thought that converting two-way operations to one-way operations would be detrimental to their businesses. Most individuals (85% of respondents) thought that one-way frontage roads were safer than two-way frontage roads.

In 2004, Texas DOT contracted with Kimley–Horn and Associates to analyze the business vitality and evaluate economic impacts after conversion of frontage roads from two-way operation to one-way operation (*11*). To meet the objective, researchers identified five study corridors in Texas. Researchers analyzed local, regional, and state economic trends, focusing on the differences between two-way and one-way conversion to examine whether economic characteristics of properties located adjacent to converted frontage roads were different. Owners of businesses along frontage roads believed that one-way frontage roads reduced traffic congestion and increased traffic safety. However, they believed that the conversion reduced access to their property. The study revealed that owners of businesses that were subjected to frontage road conversions in 2000 or 2001 were more satisfied than those whose frontage roads were converted more recently (i.e., 2002 or 2003) at the time of the study. These responses imply that business owners believe it may take customers some time to become accustomed to the frontage road conversion. Overall, it appears that there were no adverse economic impacts for businesses that were surveyed 3 years after the conversion.

Literature Review Concluding Remarks

Little previous work is published in the areas of safety and economic impacts of frontage road conversion from two way to one way. In the limited number of studies published, this review has shown that converting two-way frontage roads to one way is usually associated with a reduction in crashes, although previous studies have not used

comparison sites. In general, the results of the opinion surveys suggest that business owners and managers have concerns about reduced access to their property and associated economic impacts as a result of the conversion. It is important to ensure that enough time has passed after construction so that construction impacts are removed and business patterns have stabilized before opinion surveys are conducted.

STUDY SITES

To investigate the safety and economic impacts of conversion, researchers identified eight study sites for analysis. Table 1 summarizes the characteristics of the selected case study sites. Table 1 includes both the sites where frontage roads were converted and the case study sites used as comparison sites. The comparison sites remain two way to provide a contrasting condition to the converted sites. Researchers inspected and considered dozens of conversion sites throughout Texas. The conversion (treatment) study sites were selected to provide different highway types, geographic representation throughout Texas, and an adequate amount of economic development. All sites were converted in the past 6 years (at the time of site selection) to aid in obtaining economic information from available data sets and business owners, and all have a relatively representative comparison site. Researchers selected the comparison sites as geographically close to the conversion site as possible so that other factors that might affect safety (e.g., driver behavior, roadway design, weather) were relatively similar. Three of the pairs of conversion–comparison sites are on the same road. Table 1 also includes the number of months available for the statistical safety analysis in both the before and after periods, which is discussed in a later section of this paper.

SAFETY ANALYSIS

Crash Data Collection

Researchers obtained the electronic crash records for all crashes relevant to each study site from three sources. One source was the Texas DOT Crash Records Information System (CRIS) database, which contains all the information recorded on the peace officer crash reports from 2002 to the present, with the exception of narratives and diagrams, coded into fields. Researchers obtained electronic crash records from the CRIS database for 2003 through 2007 through requests on the Texas DOT Internet site (*12*). Texas DOT informed the research team that 2002 data were incomplete and should be omitted from the analysis.

The research team obtained crash records for 1998 through 2001 from a second source, the Department of Public Safety (DPS) historical electronic crash database. This database is composed of coded information obtained from the peace officer crash reports for crashes in Texas through the end of 2001. In 2002, management of crash reporting in Texas was transitioned from DPS to Texas DOT.

The third source of crash data was printed crash reports for the available years of 2001 through 2007. Therefore, for most crashes from 2003 through 2007 along the study sites, researchers had both electronic (coded) crash records and the printed crash report.

Researchers also collected historical traffic volume data for frontage road segments. The volume data were used to adjust the crash frequency data by roadway usage.

Crash Data Reduction

Reported crashes in the electronic databases are identified by roadway control section and mile point numbers. With the assistance of the staff

TABLE 1 Characteristics of Frontage Road Case Study Sites (*2*)

City or Site	Road	From	To	Length (mi)	Conversion Year	Before Months[a]	After Months[b]	Average Before ADT[c]	Average After ADT[d]	ADT Percent Change[e]
Conversion Sites										
Sulphur Springs	IH-30	SH-19	SL-301	5.0	2001	44	60	2,200	1,800	−18
Huntsville	IH-45	Avenue S/Smither	FM-1791	2.8	2001	43	60	4,600	5,500	20
Wolfforth	US-62/US-82	CR-1300/Quitsna	82nd St/CR-7100	2.5	2001	36	60	2,600	3,200	23
Hillsboro	IH-35	SH-22	FM-286	0.6	2001	36	60	3,100	5,500	77
Comparison Sites[f]										
Greenville	IH-30	SH-34	Division	2.8	na	44	60	1,800	2,200	22
Huntsville	IH-45	Park Road 40	Avenue S/Smither	5.8	na	43	60	3,600	4,700	31
Plainview	IH-27	Bus. IH-27 South	Bus. IH-27 North	7.6	na	36	60	1,200	1,300	8
West	IH-35	Wiggins Road	McLennan County line	5.9	na	36	60	1,900	2,100	11

NOTE: ADT = average daily traffic; na = not applicable.
[a]Number of months used in crash analysis before conversion to one way at the conversion site.
[b]Number of months used in crash analysis after conversion to one way at the conversion site.
[c]For the "before" conversion time period, this is the average of the ADT from each frontage road segment analysis unit used in the statistical analysis. Analysis units are described in the next section of the paper.
[d]For the "after" conversion time period, this is the average of the ADT from each frontage road segment analysis unit used in the statistical analysis. Analysis units are described in the next section of the paper.
[e]Percent change from the "before" period ADT to the "after" period ADT.
[f]One comparison site, which remained two way, for each conversion site, paired as follows: Sulphur Springs–Greenville, Huntsville–Huntsville, Wolfforth–Plainview, and Hillsboro–West.

of the Texas DOT Traffic Operations Division, researchers identified the control section and mile points used to identify the boundaries of each case study location for both the DPS (1998–2001) and CRIS (2003–2007) data.

Per Texas DOT suggestion, TTI researchers requested the crash reports for both the main lanes and frontage roads to ensure that no crash reports were missed because of potential coding errors. For the entire research project, researchers received more than 7,000 printed crash reports. Researchers reviewed all crash reports and the electronic data to ensure crashes on frontage roads within the study limits were kept in the database.

The electronic DPS data (1998–2001) provided more detailed information on the position and direction of the vehicles. Such information was imperative to decipher frontage road crash locations (i.e., side of roadway and direction of travel). These variables were not in the CRIS electronic database; however, researchers obtained information on the position from the officers' crash reports with the location drawings and narrative.

In 2003, there was a change in what constituted a property damage–only (PDO) crash. This change resulted in more PDO crashes in the database after 2003. Because of this change in PDO crashes in the trend analysis, researchers analyzed only non-PDO crashes, including fatalities, incapacitating injuries, nonincapacitating injuries, and possible injuries.

During the data reduction along three sites, researchers found that the variable identifying whether the crash occurred on the main lane or frontage road for the electronic crash records for 2005 through 2007 was miscoded as "mainlane," when it should have been "frontage road," approximately 36% of the time (74 of 205 crashes). Without the printed report to confirm the details of the crash, the data set might have missed crashes that actually took place on the frontage roads. For the same three study sites, researchers did not identify this problem for the DPS database in 2001. Similarly, Gattis et al. also found that between 20% and 40% of crashes were listed incorrectly in their summary database (*8, 9*). These experiences indicate the value of carefully reviewing crash reports when performing crash studies.

Safety Analysis Methodology

Researchers separated each study site into several analysis units. Researchers defined analysis units as segments (between interchanges) or interchange intersections (between traveled way of intersecting street and frontage road). Researchers kept crashes in the database if they included at least one vehicle on the frontage road and assigned them to the appropriate analysis unit.

Crashes during the construction period of the frontage road conversion were not included in the analysis. In Sulphur Springs and Huntsville, the construction period included several months in the fall of 2001. For Wolfforth and Hillsboro, data for the entire year of 2001 were removed because an exact date of conversion in 2001 could not be identified. All available data before and after these construction periods were used in the analysis. See Table 1 for the number of months available for analysis in each time period. For the comparison sites, the before and after period were the same as the associated conversion (treatment) site.

The safety effectiveness of frontage road conversion was determined by reviewing crash data before and after a conversion took place. The statistical method used to evaluate the effectiveness of the conversion was a before-and-after evaluation with comparison groups. The comparison group method assumes that the change in crashes between the before and after periods for a comparison group is representative of the change in crashes that would have occurred for the corresponding treatment group had the conversion at the treatment site not been made. It can account for the change in crashes caused by extraneous factors (e.g., weather, driving behavior, reporting practice) between the before and after periods in the safety effectiveness estimate.

One popular measure of safety effectiveness is the safety effectiveness index (θ), which is the ratio of the expected number of crashes after the treatment to what it would have been without the treatment. The safety effectiveness index (θ) was adjusted by traffic volume changes from the before to the after periods. This volume-adjusted value is reported as $\hat{\theta}_T$ in the subsequent analysis. The percent crash reduction of the treatment (conversion) (PCR_T) can be computed as $100(1 - \hat{\theta}_T)$. The weight (w_T) for each treatment group is also computed with the comparison group method accounting for traffic volume differences between before and after periods, and it is the weight for each conversion–comparison group used in calculating the overall PCR_T.

The interested reader is encouraged to review work by Hauer (*5*) and Harwood et al. (*13*) or the full research report by Eisele et al. (*2*) for more details about the statistical methods used.

Results of Comparison Group Statistical Safety Analyses

Segment Crash Results

Researchers performed statistical analyses for segments and interchange intersections separately. The results for the segment analysis are presented first. All segment results are based on segment analysis units with nonmissing traffic volumes. The segments include a variety of exit and entrance ramps, access densities, lengths, and geometric characteristics. The projects on which the percent crash reduction values are based included other improvements, such as ramp improvements, in addition to the frontage road conversion. Therefore, the percent crash reduction values inherently include some impact of the ramp configurations that occur with the conversion. It was not possible to separate these effects.

As shown in Table 2, researchers found a statistically significant 57% reduction in the expected total number of non-PDO crashes along the segments converted from two way to one way. An additional power of the comparison group method is that this particular result is based on the weighted values of each individual conversion–comparison site. The confidence intervals are provided for $\hat{\theta}_T$ in Table 2, and if the computed confidence interval does not include 1.00, the results are statistically significant.

As shown in Table 2, the annual crash frequency per year goes down from the before to after periods for all of the conversion (treatment) sites for the total of non-PDO crashes. As for the comparison sites, the number of crashes per year goes down for West and Huntsville, and the number of crashes per year goes up for Greenville and Plainview. The statistically significant percentages in the right-most column (bold) include consideration of the volume data. Traffic volumes generally increased at all sites except Sulphur Springs. In Sulphur Springs, there was an 18% average reduction in traffic volumes, and volumes increased as high as 77% at the Hillsboro conversion site.

TABLE 2 Safety Effectiveness of Frontage Road Conversion for Segment Crashes: All Non-PDO Crashes by Crash Type and Severity (2)

Conversion–Comparison Pair	BT	AT	BC	AC	$\hat{\theta}_T$	w_T	PCR_T (%)
Total Non-PDO Crashes							
Hillsboro–West	1.7	0.6	2.3	1.6	0.30	1.35	70
Huntsville–Huntsville	19.0	3.4	11.7	3.6	0.52	6.75	48
Sulphur Springs–Greenville	14.5	6.4	10.9	13.2	0.38	10.32	62
Wolfforth–Plainview	1.3	0.4	7.3	10.4	0.70	0.75	30
Overall	na	na	na	na	0.43 (0.29, 0.68)	na	**57**
Non-PDO Angle Crashes							
Hillsboro–West	0.3	0.0	0.3	0.0	na	na	na
Huntsville–Huntsville	4.2	0.2	4.2	0.6	0.32	0.69	68
Sulphur Springs–Greenville	2.2	0.4	1.6	2.6	0.11	1.07	89
Wolfforth–Plainview	0.3	0.0	0.7	0.8	na	na	na
Overall	na	na	na	na	0.17 (0.04, 0.72)	na	**83**
Non-PDO Rear-End Crashes							
Hillsboro–West	1.3	0.2	0.3	0.2	0.17	0.32	83
Huntsville–Huntsville	8.1	2.0	2.5	1.4	0.43	2.74	57
Sulphur Springs–Greenville	5.7	1.8	4.4	6.0	0.20	3.53	80
Wolfforth–Plainview	0.3	0.0	3.0	5.2	na	na	na
Overall	na	na	na	na	0.27 (0.13, 0.58)	na	**73**
Incapacitating Injury, Non-Incapacitating Injury, and Fatal Non-PDO Crashes							
Hillsboro–West	0.0	0.2	1.3	0.6	na	na	na
Huntsville–Huntsville	7.3	0.4	5.3	1.6	0.16	1.42	84
Sulphur Springs–Greenville	7.9	3.6	5.7	7.0	0.39	5.57	61
Wolfforth–Plainview	0.3	0.2	4.3	3.0	na	na	na
Overall	na	na	na	na	0.32 (0.15, 0.68)	na	**68**

NOTE: BT = annual crash frequency in the before period at the conversion (treatment) site; AT = annual crash frequency in the after period at the conversion (treatment) site; BC = annual crash frequency in the before period at the comparison site; AC = annual crash frequency in the after period at the comparison site. Bold implies statistical significance at the 95% confidence interval. The confidence intervals for the safety effectiveness index are provided in parentheses. Comparison group analysis does not consider the effect of a conversion–comparison pair when either the before or after period has zero crashes. When this occurs, the columns are indicated as "na" (not applicable).

Researchers investigated a number of crash types and crash severity. The results for non-PDO angle crashes and non-PDO rear-end crashes are shown in Table 2, along with the results of crash severity for all incapacitating injury, nonincapacitating injury, and fatal non-PDO crashes combined. As shown in Table 2, researchers also found statistically significant results for segments for

- Non-PDO angle crashes (83% reduction overall),
- Non-PDO rear-end crashes (73% reduction overall), and
- Incapacitating injury, nonincapacitating injury, and fatal non-PDO crashes (68% reduction overall).

Performing similar analyses on crash types and severity, researchers found the following statistically significant results for segments; these results are documented elsewhere in more detail (2):

- Non-PDO opposite-direction crashes (96% reduction overall),
- Non-PDO nonincapacitating injury crashes (68% reduction overall), and
- Non-PDO possible injury crashes (54% reduction overall).

Interchange Intersection Crash Results

Researchers separated crashes that were in the physical area of the interchange intersections from segment crashes. Researchers also investigated the development of safety effectiveness estimates for different types of crashes within these interchange intersections. The crash was classified as an intersection crash if the first point of collision took place within the physical area of the intersection. Because an estimate of traffic volume entering the interchange intersections was not available and could not be reasonably estimated with available traffic volume data, the percent crash reduction values for the intersection crash analysis were not adjusted by traffic volume.

For interchange intersections, researchers found the following statistically significant results for crash types and severity; these results are documented elsewhere in more detail (2):

- Non-PDO opposite-direction crashes (80% reduction overall),
- Non-PDO opposite-direction crashes including a left turn (85% reduction overall), and
- Non-PDO possible injury crashes (87% reduction overall).

Summary of CMFs and Sample Application

A CMF is a multiplicative factor used to compute the expected number of crashes after implementing a given countermeasure at a specific site. The CMF is multiplied by the expected crash frequency without the treatment. If the CMF is greater than 1.0, there is an expected increase in crashes, whereas a value less than 1.0 indicates an expected reduction in crashes after implementation of a given countermeasure (*14*).

Similar to CMFs, crash reduction factors (CRFs) are sometimes used. A CRF is the estimated percent crash reduction of some specific countermeasure and corresponds to PCR_T. The term CRF has historically been used, and it includes the word "reduction" because there was an expectation that improvement would result in reduced crashes. Sometimes, however, a reduction is not found, and the term CMF gained popularity. Several studies document the CRF calculation process and its application (*15, 16*).

If a CRF is given, a CMF is computed as 1 – (CRF/100). A CMF is the estimate of the index of effectiveness ($\hat{\theta}_T$), and Table 3 shows CMFs associated with the percent change values discussed previously. Additional CMFs are shown in Table 3; these CMFs were also estimated in the research and documented elsewhere (*2*). Table 3 provides the corresponding uncertainty estimates (95% confidence intervals) for CMFs in parentheses.

The CMFs shown in Table 3 can provide roadway designers and transportation decision makers an estimate of the possible safety impacts of converting frontage roads from two way to one way to guide frontage road conversion project planning. As an example, consider that there are 32 total non-PDO crashes on average per year, over the past 3 historical years, experienced along a particular segment of frontage road that is currently two way. Practitioners can multiply this expected number of crashes without the conversion (32) by the CMF of 0.43 (Table 3) to estimate the expected crashes with conversion ($32 \times 0.43 = 13.8$). In this example, the range is from 9.0 (32×0.28) to 21.8 (32×0.68) on the basis of the confidence interval given in Table 3. It can be concluded that with 95% confidence the true expected crashes per year will be between these confidence limits.

TABLE 3 Summary of Segment and Interchange Intersection CMFs Related to Frontage Road Conversion from Two-Way to One-Way Operation (*2*).

Item of Interest	Crash Modification Factor (CMF) Value (Confidence Interval)
Segment Crashes by Crash Type	
Total non-PDO crashes	0.43 (0.28, 0.68)
Non-PDO opposite-direction crashes	0.04 (0.00, 0.09)
Non-PDO angle and opposite-direction crashes including a left turn	0.06 (0.01, 0.33)
Non-PDO angle crashes	0.17 (0.04, 0.72)
Non-PDO rear-end crashes	0.27 (0.13, 0.58)
Segment Crashes by Crash Severity	
Non-PDO non-incapacitating injury crashes	0.32 (0.14, 0.76)
Non-PDO possible injury crashes	0.46 (0.25, 0.84)
Non-PDO incapacitating injury, non-incapacitating injury, and fatality crashes	0.32 (0.15, 0.68)
Interchange Intersection Crashes by Crash Type	
Non-PDO opposite-direction crashes	0.20 (0.05, 0.73)
Non-PDO opposite-direction crashes including a left turn	0.15 (0.04, 0.62)
Non-PDO angle and opposite-direction crashes including a left turn	0.23 (0.07, 0.75)
Interchange Intersection Crashes by Crash Severity	
Non-PDO possible injury crashes	0.14 (0.03, 0.73)

NOTE: Includes only non-PDO crashes. All values are statistically significant. An uncertainty estimate (confidence interval) associated with each CMF is provided in parentheses. See paper text and the full research report (*2*) for full methodology, assumptions, and site characteristics on which these CMFs are computed.

ECONOMIC ANALYSIS

The second objective of the research was to develop accurate information that can be used to communicate the types of economic impacts that have been experienced and can be expected when converting frontage roads from two way to one way.

Researchers investigated several data sources to estimate the economic impact of converting two-way frontage roads to one way. These sources include gross sales data from the Texas Comptroller of Public Accounts, appraisal values from county appraisal districts, employment statistics from the U.S. Census Bureau, and business owner and customer surveys. Gross sales data were available at the city and county level, and employment data were available at the zip code level. Researchers intended to compare these relatively regional data sources to responses from business owners, but insufficient survey data were obtained from business owners about objective gross sales and employment information to compare with the surveys. Therefore, the most insightful data available at the parcel level were the appraisal data and surveys from business owners and customers. For these reasons, this paper focuses on these data sources for the remainder of the discussion.

Appraisal Data Investigation

Appraisal Data Collection and Data Reduction

Land owners adjacent to frontage roads that may be converted from two-way operation to one-way operation are often concerned about the impact on property value. Fears about property value decreasing is a typical concern expressed at public meetings about potential frontage road conversion projects. To gain insight into these concerns, researchers collected appraisal data for parcels adjacent to all of the conversion and comparison sites.

For each conversion and comparison site, researchers contacted the county appraisal districts to obtain land and improvement values for parcels adjacent to the Interstates or highways studied. Specifically, researchers asked all appraisal districts for key data elements, including parcel identification, address, legal description, doing business as name, owner name, year, land value, improvement value, and total appraised value when this information was not available online. Ideally, researchers desired appraisal data from 3 to 5 years before the conversion year up to the most current year of data available.

Appraisal Data Analysis

Researchers developed graphs for the years of data available; the graphs show the appraised value in each year. In many cases, some

parcels of appraisal data were not available for a given parcel throughout the entire time series. For example, a parcel that is subdivided from another, "parent," parcel will appear in the trend as having a land value when the parcel comes under new ownership. Often, county appraisal districts indicated that it was difficult to trace the history of a particular parcel to identify the parent track, especially because parcels can be subdivided and then subsequently subdivided again. Researchers removed these parcels from the analysis. More than likely, there is physical property included in the earlier years (before the property was subdivided) that is not included in the later years. Therefore, this process provides a conservative estimate in later years of the appraised values (i.e., values are likely higher in later years than reflected in this analysis). Researchers considered leaving the additional parcel values in the analysis, but because it was difficult to know the exact parent tract, which could even be from a property that is not adjacent to the frontage road, researchers decided to proceed with the conservative trend analysis.

Appraisal Data Findings

Researchers developed trend analysis graphics showing land appraised value, improvement appraised value, and total appraised value. Results are presented in Table 4 for total appraised value. In general, the results indicate overall increases in appraised values over the historical trends investigated for all of the conversion and comparison sites. Along nearly all case study corridors, some parcels were removed from the data set because they were likely subdivisions; therefore, these percent change values should be considered conservative. From the aggregate appraisal data documented here, there do not appear to be substantial overall negative effects on appraisal values along conversion sites in the long term.

TABLE 4 Overall and Average Percent Change per Year of Total Appraised Values for Study Sites

City or Site	Years of Available Data	Percent Change[a, b]	Average Percent Change per Year[b, c]
Conversion Sites			
Sulphur Springs	2000–2010	+24	+2
Huntsville	2001–2010	+32	+4
Wolfforth	2003–2008	+84	+17
Hillsboro	2001–2010	+219	+24
Comparison Sites			
Greenville	2001–2008	+70	+10
Huntsville	2000–2010	+154	+15
Plainview	2005–2008	+19	+6
West	2006–2010	+72	+18

[a]Percent change is computed from the first year of available data to the last year of available data.
[b]It is misleading and inappropriate to compare percent change values from a conversion site with a comparison site because it is not a consistent ("apples-to-apples") comparison because of the site-specific differences produced by subdivisions of property. See paper text and elsewhere (*2*) for more details.
[c]Average percent change per year is computed as the percent change divided by the number of years of data. It was relatively common for total appraised values to change substantially from year to year, rather than at these average (consistent) levels from year to year.

Opinion Surveys of Business Owners and Managers and Customers

Opinion Survey Sample Size and Administration

Members of the business community often have concerns that frontage road conversions will negatively affect their businesses. To identify their concerns and to assist with the development of mitigation strategies, researchers surveyed business owners and managers and customers along both one-way and two-way frontage roads. Researchers conducted 80 surveys of business owners and managers and 74 customer surveys at all sites. Researchers conducted 36 of the business owner and manager surveys and 49 of the customer surveys at businesses along one-way frontage roads.

Before conducting the business owner and manager and customer surveys, the research team performed a windshield survey by driving along the selected corridor, videotaping, diagramming, and noting useful information. Researchers called all businesses along the corridor in an attempt to make appointments with business owners and managers to survey them. If a business participated in the business survey, it was later contacted to determine the willingness of the owner or manager to have the business participate in the customer survey.

Researchers read questions to the business owners and managers, and a member of the research team recorded their response. There were 22 questions in the survey at conversion sites and 21 questions at comparison sites. Each survey contained four open-ended questions.

Customer surveys were handed out to the customers by a member of the research team or left at the business to be handed out by the business owners and managers. There were 12 questions in the customer survey along conversion sites and 10 questions along comparison sites. There was one open-ended question at the end of the survey.

Most questions on all four surveys required a response (i.e., "no response" was not an option), but respondents would skip questions if they were not sure of the answer. Inherently, most of the survey responses resulted in subjective perception data.

The interested reader is encouraged to review the full report to see the complete survey instruments (*2*).

Survey Data Reduction and Statistical Analysis Methods

Researchers conducted chi-square tests to determine if there were statistically significant differences in responses for a number of analyses. If the expected count was less than five for more than 20% of all cells, researchers performed a Fisher–Freeman–Halton exact test instead of a chi-square test. Researchers performed the test using StatsDirect (*17*). Researchers investigated two significance levels, p-values of 0.05 and 0.01, in all cases.

Opinion Survey Findings

Researchers conducted a number of statistical analyses on the survey data. This section highlights the results related to customer likelihood of stopping at businesses and several potential operational and economic impacts.

Opinions on Customer Likelihood of Stopping at a Business

Customers were asked the following question: "If the frontage road

were converted, would you be more likely, less likely, or have the same likelihood of stopping at this business?" Researchers computed results relative to whether it was a conversion or comparison site, whether they had to leave in the opposite direction to get where they were going, and whether they had to loop around the interchange to get to an establishment on the opposite frontage road. Statistically significant differences were found. Results include the following:

- Along conversion sites, 29% of customer survey respondents indicated that they would be less likely to visit the business and 67% said the likelihood would be the same, as opposed to 71% at comparison sites indicating less likely and 29% indicating the same.
- Of customers who had to leave the business in the opposite direction of desired travel, 61% indicated they would be less likely to visit the store if the roadway operation were one way.
- Of customers who had to make a loop around the interchange to get to a store on the opposite frontage road, 55% indicated they would be less likely to visit the store if the roadway operation were one way.

Opinions on Operational and Economic Impacts The next analysis investigated a number of operational and economic factors asked of business owners and managers about frontage road conversion. Statistically significant results were found related to property values, gross sales, and delivery convenience. The key findings include the following:

- Along conversion sites, 64% of respondents indicated they thought property values had increased, whereas along comparison sites, 21% expected property values to increase.
- Along conversion sites, 76% of respondents indicated they believed gross sales were down along the conversion site, whereas 52% expected reduced gross sales at comparison sites if the frontage roads were ever converted.
- Eighty-four percent of retail business owners and managers believed delivery convenience would be worse, whereas 51% of non-retail business owners and managers believed delivery convenience would be worse.

Researchers hypothesized that the bias of business owners and managers against one-way frontage roads may affect these results. For this reason, they asked business owners and managers, "How do you classify your preference for one-way compared to two-way traffic on freeway frontage roads in urban areas?"

Researchers compared responses to all economic and operational categories previously investigated. Table 5 shows the statistically significant differences on the basis of business owners' and managers' preference for frontage road operation. Business owners and managers who indicated they either strongly or somewhat favor one way are in the one-way group (row) of Table 5. If they indicated they either strongly or somewhat favor two way, they are in the two-way group (row). The results in Table 5 clearly indicate the myriad concerns of business owners and managers who favor two-way frontage roads about the number of customers visiting their businesses, as well as perceived degradation of business opportunities, customer satisfaction, property access, and delivery convenience. Meanwhile, business owners and managers who are characterized as preferring one-way frontage roads acknowledged less traffic congestion, traffic safety, and decreased crashes.

Concluding Remarks on Opinion Surveys

The survey findings are from small sample sizes. Further, the survey data are opinions and perceptions of customers and business owners and managers. Researchers were not able to obtain actual gross sales or employment data. With these caveats in mind, researchers offer the following conclusions and comments from the survey responses and analysis:

- From open-ended responses, there is interest from the business community in doing conversions as quickly as possible once construction starts and constructing the support infrastructure before the frontage road conversion (bridges, turnaround locations, signage). Identifying opportunities for property access (e.g., cross access, shared access, alternate streets) along the two-way frontage road before the frontage road conversion is important. It is also important to engage all stakeholders and the business community early and often regarding a possible project.
- Customers would rather not travel in the opposite direction when leaving a business and may be less likely to stop at a business if travel in the opposite direction is necessary.
- Business owners and managers seem concerned about the number of customers they will receive and possible reductions in

TABLE 5 Statistically Significant Differences Between Preferences of Business Owners and Managers for One-Way or Two-Way Frontage Roads (2)

Group	Decrease	No Change	Increase
Number of Customers? (p-value = .005)			
1-way	28%	44%	28%
2-way	64%	18%	18%
Property Value? (p-value = .018)			
1-way	14%	52%	33%
2-way	38%	20%	43%
Number of Crashes? (p-value = .002)			
1-way	44%	52%	4%
2-way	20%	46%	35%

Group	Worse	Same	Better
Traffic Congestion? (p-value = .007)			
1-way	15%	12%	73%
2-way	33%	33%	33%
Traffic Safety? (p-value = .007)			
1-way	8%	15%	77%
2-way	27%	44%	29%
Property Access? (p-value = .007)			
1-way	54%	27%	19%
2-way	88%	8%	4%

Group	Worse	Same	Better
Business Opportunities? (p-value = .033)			
1-way	28%	60%	12%
2-way	60%	32%	9%
Customer Satisfaction? (p-value = .037)			
1-way	36%	60%	4%
2-way	60%	29%	10%
Delivery Convenience? (p-value = .001)			
1-way	32%	40%	28%
2-way	73%	25%	2%

NOTE: Differences are statistically significant. Shading indicates notable differences. Percentages may not add up to 100% because of rounding.

gross sales after conversion. This concern is despite indications of their property value increasing.

- When business owners and managers are asked if they generally prefer one-way or two-way frontage roads, there is a connection between their preference and their concerns. Business owners and managers who prefer one-way frontage roads generally indicate that they believe there is a decrease in crashes, increase in traffic safety, and improvement in traffic congestion.
- Most business owners and managers believe that the frontage road conversion will harm their business. In general, their concerns revolve around access, gross sales, customer satisfaction, construction, ramp and interchange locations, and freeway signage. It appears that the perceived economic impacts may be related to timing of additional infrastructure placement and construction (open-ended responses).
- Retail business owners and managers are more concerned about delivery convenience than nonretail business owners and managers.

CONCLUSIONS

In this paper, researchers describe research performed to estimate the safety and economic impacts of converting two-way frontage roads to one way. Researchers investigated four converted study sites and four comparison study sites that remained two way.

Safety Impacts

To satisfy the objective related to developing valid information about safety impacts, researchers developed 12 statistically significant CMFs related to frontage road conversion for segments and interchange intersections. The CMFs are associated with various crash types and crash severity. Researchers found an overall 57% reduction in all non-PDO crashes (CMF = 0.43). Researchers describe how the CMFs were developed and associated caveats and provide an example application of the CMFs. The CMFs all indicate a reduction in crashes with the conversion, even with increasing traffic volumes, and this finding is consistent with the few studies described in the literature. In contrast to previous research, the research described here includes the use of comparison sites to account for the change in crashes caused by extraneous factors (e.g., weather, driving behavior, crash reporting practice).

Economic Impacts

The second objective of the research was to develop accurate information that can be used to communicate the types of economic impacts that have been experienced and can be expected when frontage roads are converted from two-way to one-way operation. The most insightful data available at the parcel level were the appraisal data and surveys from business owners and managers and customers. Researchers developed trend analysis graphics showing land appraised value, improvement appraised value, and total appraised value (i.e., real property value). The appraisal value results indicated an overall increase in appraised values over the historical trends investigated at all conversion and comparison sites. From the appraisal data documented here, there do not appear to be substantial overall negative effects on appraisal values along conversion sites in the long term.

Researchers also surveyed the opinions of business owners and managers and customers. In general, most business owners and managers believed the frontage road conversion would harm their business. Similar responses from the business community were documented in previous research (*8–11*). In general, concerns of business owners and managers revolve around access, gross sales, customer satisfaction, ramp and interchange locations, and freeway signage. From the responses to the open-ended questions, it appears that perceived economic impacts may be related to timing of infrastructure placement and construction, especially interchanges that will decrease the distance required to loop around to establishments on the opposite frontage road. These concerns point to the importance of identification by public agencies of opportunities for property access (e.g., cross access, shared access, alternate streets) along the two-way frontage road before the frontage road conversion. It is also important to engage all stakeholders and the business community early and often regarding a possible project.

Crash Data Lessons Learned

The final objective of this research was to identify issues and lessons learned from the crash data for those performing similar safety analyses. Researchers noted that nearly 40% of the crash records along three of the sites were incorrectly coded as taking place on the main lane when they actually took place on the frontage road. As a result, researchers had to manually verify all printed crash reports (main lane and frontage road) to ensure that all applicable frontage road crashes remained in the analysis. Previous research documented similar experiences (*8, 9*). Researchers also identified that critical position variables, used to indicate the side of the road a vehicle was traveling on and in what direction, were removed from the electronic database for 2002 and later years. Researchers manually created and filled these variables into the final electronic database used for analysis by reviewing the diagrams on the crash reports. Researchers also were informed of a change in how PDO crashes were determined in 2003 and that different base maps (and subsequently different control section and mile point numbers) were used for crash data before and after 2001. These experiences indicate the importance of fully understanding all nuances in the crash data set prior to analysis. Not adjusting for any one of these items would have produced misleading results.

Future Work

There is a need to expand the frontage road analysis performed here to additional sites. Researchers were limited by traffic volume not being available for some of the frontage road segments, and volume data not being available for any of the interchange intersections. Additional sites and more traffic data would facilitate more powerful statistical techniques (e.g., empirical Bayes methods), which may provide even more robust estimation of frontage road conversion CMFs.

Additional sites would also benefit the economic analysis. Additional conversion sites would provide a greater sample of opportunity for surveys of business owners and managers and of customers as well as more opportunity to investigate gross sales, appraised value, and employment impacts of conversion. In future work, it would be valuable to investigate the possibility of obtaining corridor-specific gross sales data, if available.

Regarding the property value analysis, it is important to note that this study evaluated real property value using appraisal data. When business owners and managers express concern over value, it is typically the going concern value (or business enterprise value); this appraisal term encompasses more than just the real value and includes elements of the business operation itself. Future economic analysis should investigate this distinction further, if possible.

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Analytical Capacity Comparison of One-Way and Two-Way Signalized Street Networks

Vikash V. Gayah and Carlos F. Daganzo

Recently cities have been converting traditional one-way downtown street networks to two-way operation partly because one-way networks are seen as confusing and as less conducive to economic activity and a livable environment and they require vehicles to travel longer distances on average. However, one of the main disadvantages of such conversions is thought to be a reduction in the network's ability to serve vehicles. Intersections in two-way networks can serve fewer vehicles per unit time than their one-way counterparts. Several studies have assessed the differences between these two types of networks, but most studies are site specific and do not consider the best possible two-way networks. This paper presents an analytical model that uses macroscopic analysis techniques to compare various one-way and two-way networks using their trip-serving capacities. This metric is a key indicator of network performance. Two-way networks can serve more trips per unit time than one-way networks when average trip lengths are short. This study also found that two-way networks in which left-turn movements were banned at intersections could always serve trips at a higher rate than one-way networks could, even long trips. Thus, the trip-serving capacity of a one-way network can actually be increased when it is converted to two-way operation if left turns are banned. In this way, livability and efficiency objectives can be achieved simultaneously. This framework can be used by planners and engineers to determine how much a network's capacity changes after a conversion, and also to unveil superior conversion options.

In the 1920s and 1930s, before private automobiles started to gain prominence in the United States, most traffic networks in urban downtown areas were designed as a system of one-way streets. City blocks were generally short and streets narrow, and one-way streets were an attempt to accommodate cars in urban areas that were not originally designed for them. In time, as the private automobile became the dominant mode and cities began planning around it, one-way operation was generally kept in downtown areas because of the conventional wisdom that one-way streets offered higher average speeds, easier signal progression, and greater capacity to move vehicles (*1*).

Within the past 20 years, many cities have started to explore converting a portion or all of their downtown streets from one-way to two-way operation (*2*). Many reasons are cited for the preference of two-way streets (*3*), but there is much debate about the issue. Two-way streets are believed to be more friendly to pedestrians and conducive to economic development. Although some argue that the opposite may be true because one-way streets may offer higher vehicle flows and, therefore, less congestion (*4, 5*), others believe that some amount of congestion may be a sign of a healthy, economically thriving street (*6*). One-way streets are sometimes viewed as less safe than two-way streets because of faster travel speeds and increased driver inattention (*7*). However, one-way streets create fewer conflicting maneuvers at intersections (*8, 9*), and one-way streets in central business districts have been empirically shown to have fewer collisions (*10*).

One-way streets are also criticized because they are more confusing for users, especially those using public transportation, because return trips often cannot originate on the same street on which the outbound trip ends. Another criticism is that one-way streets force drivers to take circuitous routes, which increases average trip lengths and, therefore, the total vehicle miles traveled in the central business district. Circuitous routes are especially problematic for emergency vehicles, which need to reach their destination quickly, and transit vehicles.

Most studies appear to agree that one-way street networks offer higher vehicle-moving capacities than two-way networks. This increased capacity is because two-way networks allow conflicting left-turn maneuvers at intersections, and efforts to mitigate these conflicts tend to reduce vehicle flows through the intersection. For example, separate phases can be added to the intersection signal for left-turn movements, but these phases increase lost time at the intersection. Additional lanes can also be used to keep left-turn queues from blocking other vehicles, but these lanes reduce the space available for through vehicles to discharge. Thus, two-way networks allow fewer lanes to discharge at once, which reduces the maximum vehicle flow that can be served. However, a reduced capacity to serve vehicle flows does not necessarily translate into a reduced capacity to serve vehicle trips.

Perhaps because of this disconnect, the efficiencies of one-way and two-way networks are still being debated. Some design handbooks attempt to theoretically quantify the change in vehicle-moving capacity from a conversion. Estimates range from 10% to 50% (*9*). However, these numbers are calculated from corridor-level analyses that are not able to account for the rate at which trips are served. Other studies examine changes in traffic operations from local conversions involving just a few streets or blocks. These studies (*11–16*) use empirical data, microsimulation of a small portion of the network, and, in some cases, heuristic rerouting schemes to quantify the changes. However, the results tend to be site specific, so generalizations to other networks cannot be made. Furthermore, these studies consider only a narrow spectrum of conversion possibilities.

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Macroscopic modeling techniques seem well suited to compare the capacities of different networks. It has recently been shown with both theory and empirical data that a unique, reproducible relationship exists between the average flow and the average density of redundant networks covering neighborhoods if trips are roughly uniformly distributed (*17, 18*). This relationship occurs because drivers tend to distribute themselves evenly across all links in the network as they adapt to local pockets of congestion (*19, 20*). If the average trip length is stable over time, the average flow–density relationship also describes a relationship between the rate that trips are completed and the level of congestion in the network. The maximum trip completion rate (or trip-serving capacity) is a key determinant of how a network performs during a rush hour (*17*). This maximum rate systematically occurs when the network is uncongested but on the verge of oversaturation (*21*). This last reference also shows that if blocks are sufficiently long, the trip-serving capacity of the network is not affected by the signals' offsets.

The objective of this paper is to assess the relative trip-serving capacities of various one-way and two-way networks. An analytical model is used to directly estimate the change in trip-serving capacity expected as a result of a networkwide conversion. The insights gained from this approach may be useful to planners and engineers contemplating large-scale conversions, or even considering local changes. This paper also presents a systematic comparison of various networks to determine which can serve the most vehicle trips per unit time. This comparison shows that two-way networks in which left turns are banned at intersections provide higher trip-serving capacities than one-way networks and other two-way networks.

The authors describe the idealized network and the capacity formula that is used in this study, describe the networks and compare their trip-serving capacities, describe how the results may change when some simplifying assumptions are relaxed, and summarize the main findings.

CAPACITY FORMULA

Considered here is an idealized grid network with long blocks that is completely homogeneous (i.e., all links are the same). It is assumed that origins and destinations are evenly spread across all links so that travel is evenly split between the two major travel directions (without loss of generality, these directions are assumed to be north–south and east–west), and that the average trip length is known and stable over time.

The analysis focuses on the operation of the intersections because the trip-serving capacity of a street network is mainly constrained by the individual intersections if blocks are long (*21*). It is assumed that all intersections are signalized, that the available green time is evenly split between the north–south and east–west directions, and that the lost time between consecutive phases is constant. It is also assumed that left turns are allowed only during protected left-turn phases (if possible) and that these left-turn phases are perfectly timed to serve the left-turn demand with no wasted time. This last assumption is optimistic but will provide an upper bound on the trip-serving capacities of two-way networks that allow left turns. The effects of relaxing this and other optimistic assumptions will be discussed later.

The authors start the analysis by envisioning a network made up of two-way streets with intersections that have wide lanes and overpasses with ramps to facilitate turning movements, so that conflicting movements are physically separated, as shown in Figure 1. Further, the authors assume that travel time on the ramps is negligible. Because the number of vehicles able to move through an intersection is proportional to the product of the number of lanes and available green time, measured in lane seconds, an intersection operated as in Figure 1 should serve the highest vehicle flows because vehicles are able to discharge continuously using all lanes. In addition, because two-way streets are used and all turn movements are allowed, vehicles are able to take the most direct routes to reach their destinations. This configuration allows trips to be completed at the highest rate possible. More realistic networks with signalized intersections instead of overpasses should have lower trip-serving capacities for four main reasons: (*a*) red phases at the intersection signals would prohibit vehicles from discharging during a portion of the cycle; (*b*) physical separation of through and left-turn movements would mean fewer lanes are able to discharge at once; (*c*) narrower lanes provided to accommodate left-turn pockets would prohibit vehicles from discharging at the maximum rate; and (*d*) one-way streets or turn restrictions (if implemented) would force vehicles to travel longer distances on average, which results in a lower trip completion rate (*17*).

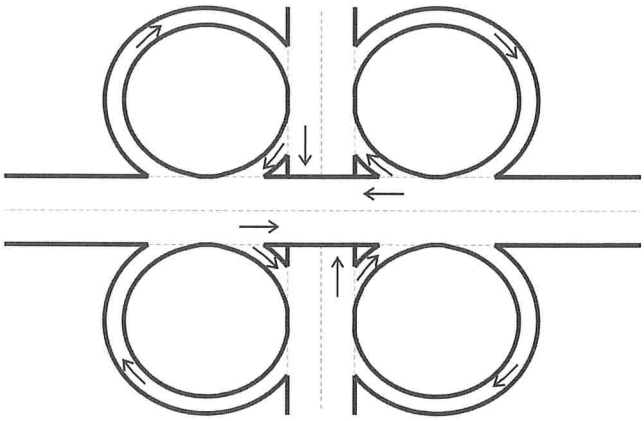


FIGURE 1 Intersection configuration for idealized network.

This idealized, unsignalized network is used as a benchmark against which to compare signalized networks. The relative trip-serving capacity of each network, denoted C_i, is given as the proportion of the maximum rate at which trips can be made using network i as compared with the maximum rate of the idealized network. For the remainder of this paper, the term "capacity" will refer to the trip-serving capacity defined in this way. This capacity can be expressed as the product of (*a*) the ratio of lane seconds of discharge time available in network i compared with the idealized network; (*b*) the ratio of saturation flows through the intersection in network i compared with the idealized network; and (*c*) the ratio of average trip lengths in the idealized network compared with network i. Because both the north–south and east–west directions are symmetric, the capacity of network i can be determined by examining the operation of an intersection as it serves one of these directions, as follows:

$$C_i = \left[\frac{N_i^T G_i^T + N_i^L G_i^L}{N_i C}\right]\frac{s_i}{\alpha_i} = \left[\frac{N_i^T g_i^T + N_i^L g_i^L}{N_i}\right]\frac{s_i}{\alpha_i} \quad (1)$$

where

C_i = relative trip-serving capacity of each network;
N_i^T and N_i^L = total number of through and left-turn lanes, respectively, available for discharge in one direction for network i;
N_i = total number of wide lanes that would be available in idealized network using same space;
G_i^T and G_i^L = green time available per cycle for through and left-turning vehicles, respectively, to discharge;
C = length of signal cycle;
s_i = ratio of saturation flows at intersection in network i compared with idealized network;
g_i^T and g_i^L = fraction of green time per cycle available for through and left-turn movements; and
α_i = ratio of average trip lengths in network i compared with idealized network.

The numerator of the term in brackets in the first equality is the lane seconds of green available per cycle to both through-moving and left-turning vehicles traveling in one direction in network i, and the denominator is the lane seconds of green that would be available to all vehicles traveling in the same direction in the idealized network. Thus, the term in brackets is the ratio of lane seconds of green available in network i compared with the idealized network. This, multiplied by the ratio of saturation flows and the inverse of the ratio of the average trip lengths, yields the fraction of trips possible per unit time using network i as compared with the idealized network. The capacity can be equivalently expressed as a function of the fraction of green time per cycle available for through and left-turn movements (denoted g_i^T and g_i^L, respectively) as shown in the second equality of Equation 1.

ANALYSIS OF NETWORKS

This section uses Equation 1 to compare the capacities of different networks, and the following subsection describes the networks. The subsequent subsection shows how the parameters of Equation 1 are determined, and the final subsection compares the capacities of the different networks.

Networks

The intersection configurations for the different networks considered in this study are shown in Figure 2. The networks are two-way network with overlapping left-turn lanes ($i = a$), two-way network with nonoverlapping left-turn lanes ($i = b$), two-way network with left-turn pockets ($i = c$), two-way network with presignals ($i = d$), two-way network with single-lane approaches ($i = e$), two-way network with banned left turns ($i = f$), and one-way network ($i = g$). Only networks that treat all links in the same way are considered.

Figure 2, *a* and *b*, shows intersection configurations for a two-way network in which an upstream lane becomes dedicated to left-turn maneuvers at the intersection. Where left-turn lanes overlap, only one lane of space is taken from through vehicles to serve left turns at opposing approaches. However, when left-turn lanes do not overlap, two lanes are taken from through vehicles to serve the left turns at opposing approaches. Clearly, the nonoverlapping case is less efficient than the overlapping case. However, networks with overlapping left-turn lanes can only be used when there are an odd number of lanes available at the intersection.

When left-turn pockets are used, as in Figure 2*c*, lanes are narrowed upstream of the intersection to create a turn pocket that left-turning vehicles use to discharge at the intersection. This pocket provides space for left-turning movements without taking away a lane from through movements, but it reduces the discharge rate at the intersection because vehicles must discharge through narrower lanes.

Figure 2*d* uses additional signals (called presignals) upstream of the intersection approach, as described by Xuan et al. (*22, 23*), to reorganize vehicles at the intersection in such a way that through-moving and left-turning vehicles are able to discharge from all lanes during their respective phases. The presignal strategy requires long blocks to accommodate the presignal queues, and this is assumed to be the case.

Figure 2*e* shows the special case of a two-way network where intersection approaches consist of one lane. Protected left-turn phases cannot be used at the signal because left-turn and through vehicles are mixed together. Instead, both must discharge during the same phase. Left-turning vehicles must wait for a gap in the opposing traffic (produced by the arrival of another left-turning vehicle) before proceeding. During this time, the waiting left turner will block all vehicles behind it. This blocking reduces the average discharge rate through the intersection.

The last two-way network, shown in Figure 2*f*, bans left-turn movements at the intersection. This movement restriction will increase average trip lengths but will also reduce the number of phases required at the intersection signal. The same is true for the one-way network presented in Figure 2*g*.

Network Parameters

This section describes how the parameters of Equation 1 are determined for each network in terms of its relevant attributes. These parameters, and therefore the capacities of each network, turn out to depend only on the number and type of lanes at the intersection, the lost time per phase change at the signal, and the average trip length in the network.

Number and Type of Lanes (N_i, N_i^T, N_i^L)

The number and type of lanes available to serve vehicles in one direction is easily determined by examining the intersection geometry. Values for each network are provided in Figure 2 for the specific intersection configurations shown.

For example, in Figure 2, network a has three lanes of space available to serve a particular direction. Assuming these lanes are wide, an idealized network using the same amount of space would have the same number of lanes, so $N_a = 3$. Examining the east–west direction, through-moving vehicles are able to discharge from two lanes: one from east to west and one from west to east. Likewise, left-turning vehicles are able to discharge from two lanes, so $N_a^T = 2$ and $N_a^L = 2$.

In network c, however, four wide lanes are narrowed to provide five narrow lanes of space at the intersection. An idealized network using the same amount of space would only be able to have four lanes, so $N_c = 4$. In the east–west direction, through-moving vehicles are able to discharge from four narrow lanes, and left-turning vehicles from two narrow lanes, so $N_c^T = 4$ and $N_c^L = 2$.

In network g, three lanes of space are available to serve each direction, so $N_g = 3$; note that only the eastbound and southbound

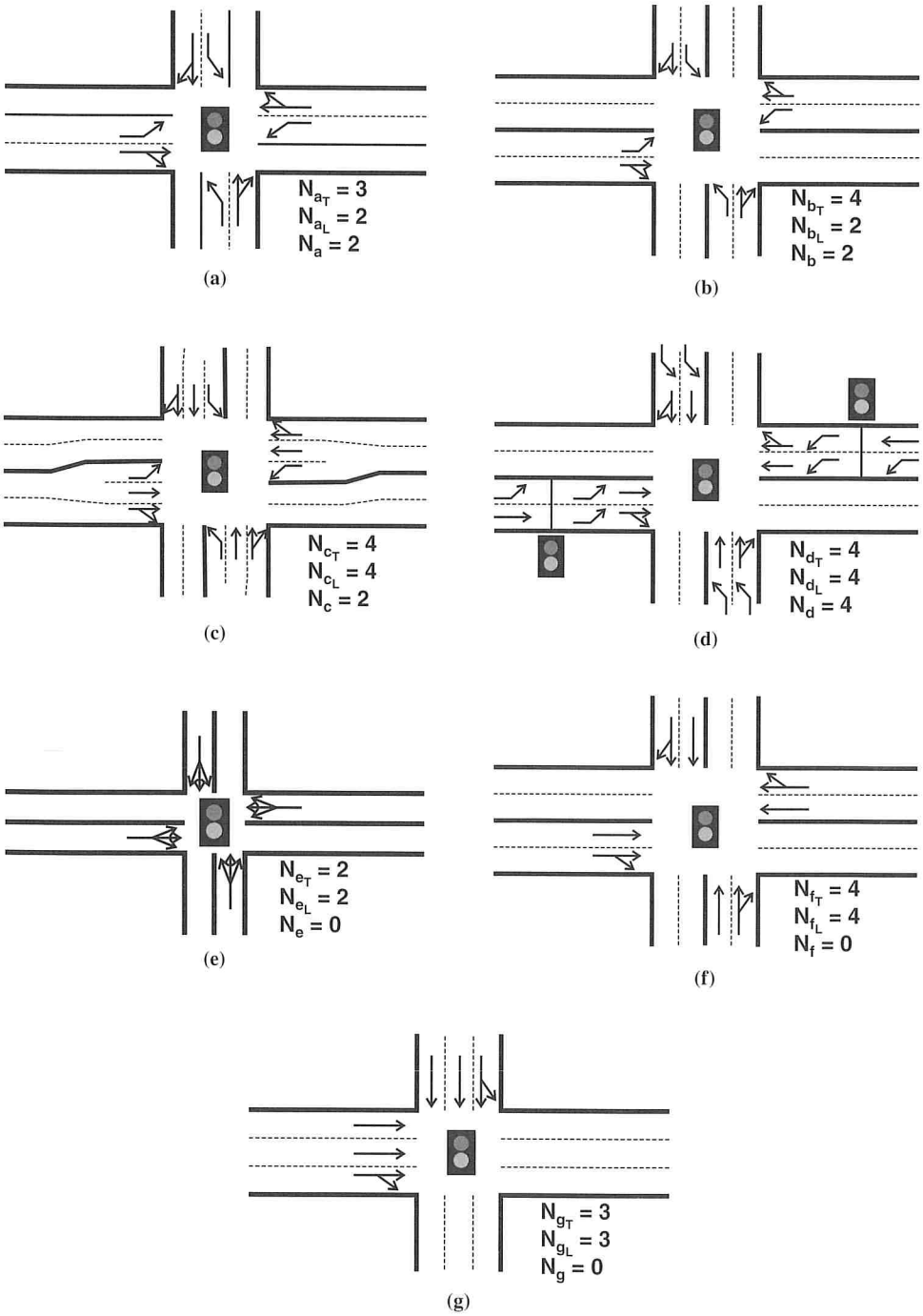


FIGURE 2 Intersection configurations for (*a*) two-way network with overlapping left-turn lanes, (*b*) two-way network with nonoverlapping left-turn lanes, (*c*) two-way network with left-turn pockets, (*d*) two-way network with presignals, (*e*) two-way network with single-lane approaches, (*f*) two-way network with banned left turns, and (*g*) one-way network.

directions are served by this intersection because streets are one-way. Because all lanes serve through vehicles, but only one way, and no left-turn lanes or movements exist at the intersection, $N_g^T = 3$ and $N_g^L = 0$.

Green Fractions (g_i^T, g_i^L)

For the first four networks presented in Figure 2, *a* through *d*, two phases are needed to serve each direction: a protected left-turn phase and a through phase. Because signals are evenly balanced between the north–south and east–west directions, half of the cycle is available to serve each. However, not all of this time can be used by vehicles to discharge because some time is lost between phase changes. If a four-phase signal scheme is used that allows a left-turn phase and a through phase for each of the two directions, four lost times will be incurred per cycle. Half of these lost times may be assigned to each direction. Thus, the fraction of the cycle available for through and left-turn phases in one direction is

$$g_i^T + g_i^L = ½ - 2l \qquad i = a, b, c, d \tag{2}$$

where l is the fraction of the cycle lost per phase change and $l << 1$.

To calculate how this green time can be optimally assigned to serve the through and left-turn phases, assume that the average trip length is n (measured in blocks traveled between the origin and destination) and the average number of left turns made per trip as a function of the trip length is $L(n)$. The ratio $L(n)/n$ is the average number of left turns made per vehicle block traveled or, equivalently, the proportion of vehicles that turn left, $P_L(n)$. If the signal is perfectly timed so that through and left-turn movements are on the verge of oversaturation, the lane seconds of green available to each must satisfy

$$\frac{g_i^L + N_i^L}{g_i^L N_i^L + g_i^T N_i^T} = \frac{L(n)}{n} \doteq P_L(n) \qquad i = a, b, c, d \tag{3}$$

Or equivalently

$$\frac{g_i^L}{g_i^T} = \frac{N_i^T}{N_i^L} \frac{L(n)}{n - L(n)} \qquad i = a, b, c, d \tag{4}$$

$L(n)$ can be calculated from the vehicle routing strategy, which is assumed to be the most direct between all origins and destinations. It is assumed that vehicles choose from among the many shortest paths available by minimizing the number of left turns. In this case

$$L(n) = \frac{6n - 2}{8n} \qquad n \geq 2 \tag{5}$$

When Equation 5 is substituted into Equation 4, the proportion of time needed for the left-turn phase decreases with n and N_i^L but increases with N_i^T.

Thus, the capacity of networks *a* to *d* can be found by substituting Equations 2 through 5 into Equation 1. The terms of this expression can then be simplified, yielding

$$C_i = \frac{½ - 2l}{N_i} \left[\frac{N_i^T N_i^L n}{N_i^T L(n) + N_i^L (n - L(n))} \right] \frac{s_i}{\alpha_i} \qquad i = a, b, c, d \tag{6a}$$

For the other three networks (*e*, *f*, *g*), left-turn phases and left-turn lanes are not needed at the intersection; the intersection can be served with a two-phase traffic signal and all lanes are available to through vehicles. The capacity of these networks simplifies to

$$C_i = (½ - l) \frac{s_i}{\alpha_i} \qquad i = e, f, g \tag{6b}$$

Saturation Flow Ratios (s_i)

For networks *a*, *b*, *d*, *f*, and *g*, vehicles are assumed to discharge from the intersection at the same rate as the idealized network that uses the same road space; thus, $s_i = 1$ for $i = a, b, d, f, g$.

For network *c*, vehicles should discharge at a lower rate because lanes are narrowed at the intersection. The rate at which vehicles discharge from these narrow lanes is determined as suggested in the *Highway Capacity Manual* (*24*). Assuming four 12-ft-wide lanes that narrow into five 9.6-ft-wide lanes, as shown in Figure 2*c*, the saturation flow fraction, s_c, is

$$s_c = 1 + \frac{w - 12}{30} = 1 + \frac{9.6 - 12}{30} = 0.92 \tag{7}$$

where w is the width of the lanes at the intersection (in feet).

For network *e*, left-turn vehicles block through vehicles at the intersection. The average saturation flow can be calculated as a function of the proportion of left-turning vehicles at the intersection, $P_L(n)$. A Markov chain was created on the assumption that two opposing vehicles can discharge simultaneously if they are making the same movement (i.e., both left turning or both through) and that only the through vehicle is able to discharge if the opposing vehicles are making different movements. The solution to this Markov chain yields

$$s_e(n) = \frac{2 - P_L(n)}{3 - 2P_L(n)} \qquad \text{for } 0 < P_L(n) \leq 1 \tag{8}$$

Equation 8 shows that the saturation fraction, s_e, decreases as $P_L(n)$ decreases, approaching ⅔ as $P_L(n)$ approaches 0. This decrease makes physical sense: as left turns become more infrequent, any left-turning vehicle that does approach the intersection must wait at the intersection (blocking all other vehicles) for a longer time on average until an opposing left-turn vehicle arrives and they can both move. Because $P_L(n) = L(n)/n$, it can be seen that the saturation fraction decreases with n for this network.

Average Trip Lengths (α_i)

For all networks except *f* and *g*, vehicles are able to use the most direct route to reach their destination because there are no restrictions on movement through the network; thus, $\alpha_i = 1$ for $i = a, b, c, d, e$.

For networks *f* and *g*, however, vehicles will have to travel longer routes on average to reach their destination because of left-turn or one-way restrictions. Using network geometry and probability theory, one can find the average additional distance traveled in networks *f* and *g*, denoted $D_f(n)$ and $D_g(n)$, respectively, as a function of the average trip length. $D_f(n)$ increases with n and approaches an upper bound of one additional block length traveled per trip, whereas $D_g(n)$ generally decreases with n and approaches a lower bound of two additional block lengths traveled per trip. Thus, the

ratio of average trip lengths in the two-way network with banned left turns as compared with the idealized network is

$$\alpha_f(n) = \frac{n + D_f(n)}{n} < 1 + \frac{1}{n} \tag{9}$$

For the one-way network, the ratio is

$$\alpha_g(n) = \frac{n + D_g(n)}{n} > 1 + \frac{2}{n} \tag{10}$$

Because Equations 9 and 10 imply that $\alpha_g(n) > \alpha_f(n)$ for all n, one-way networks always impose more circuitous routes than two-way networks with banned left turns. In both cases, the proportion of additional travel distance decreases with n; therefore, as trips become longer, the penalty imposed by circuitous routing decreases.

Comparison of Networks

Equations 6a and 6b are now applied to the different networks to evaluate their capacities. Because g_i^T, g_i^L, s_i, and α_i are functions of n and l, the capacities of each network are plotted versus n for two different values of l. Figure 3, *a* through *c*, presents the capacities when lost time between signal phase changes is negligible ($l = 0$) and Figure 4, *a* through *c*, presents the capacities when $l = 0.05$, for two-, three- and four-lane networks, respectively. Only networks *e*, *f*, and *g* can have two travel lanes; only networks *a* and *g* can have three travel lanes; and only networks *b*, *c*, *d*, *f*, and *g* can have four travel lanes.

The capacities of the two-way networks with overlapping lanes (network *a*), nonoverlapping lanes (network *b*), and presignals (network *d*) do not vary with n in Figures 3 and 4. They do not vary because in these networks vehicles are able use the same number of lanes to discharge during both the left-turn and through phases (two, two, and four, respectively). Thus, the length of the left-turn phase (which is a function of n) does not change the capacity (i.e., the total lane seconds of green available for vehicles to discharge does not change with the length of the left-turn phase). Compare this network with the two-way network with left-turn pockets (network *c*). In this network, two lanes discharge during the left-turn phase and four lanes discharge during the through phase. Thus, the capacity will increase as the left-turn phase becomes shorter (or as n increases). Also note that, as expected, increased lost time reduces the capacities of networks that use a four-phase signal scheme (*a*, *b*, *c*, *d*) more than networks that use a two-phase signal scheme (*e*, *f*, *g*).

Figures 3 and 4 show that, contrary to conventional wisdom and design handbooks, one-way networks do not always serve more trips than two-way networks. For networks consisting of two travel lanes, the two-way network with single-lane approaches can serve more trips per unit time than the one-way network when average trip lengths are relatively short ($n < 4$). For three-lane networks, the two-way network with overlapping turn lanes also outperforms the one-way network for shorter trips ($n < 4$ for $l = 0$ and $n < 3$ for $l = 0.05$). The same pattern is also seen in four-lane networks: the two-way network with left-turn pockets offers higher capacity than the one-way network for shorter trips ($n < 14$ for $l = 0$ and $n < 6$ for $l = 0.05$). Because most downtown areas are small and trips can be expected to be short, it is possible that converting a downtown network from one-way operation to two-way operation may increase its ability to serve trips, depending on the network used.

Nontraditional two-way networks can also provide superior capacity to one-way networks. The two-way network with presignals outperforms the one-way network for all trip lengths when $l = 0$ and for shorter trips when $l = 0.05$ ($n < 17$). In addition, the two-way network with banned left turns always outperforms the one-way network. This performance is because both networks provide the same number of lanes and require the same signal phasing, but the two-way network with banned left turns imposes less circuitous routes than the one-way network. Therefore, a one-way to two-way network conversion can always be used to increase capacity if left turns are simply banned at the intersections. The two-way network with banned left turns also outperforms the presignal network for long trip lengths ($n > 8$) when $l = 0.05$. As lost time increases, the benefits of using a two-phase signal outweigh the additional circuity imposed by banning left turns, and this network becomes even more effective than using presignals.

The results presented in Figures 3 and 4 assume that signal settings are the same for one-way and two-way networks. However, when a network is converted, it is likely that the cycle lengths at the intersection will be changed. In this case, the model can still be used to compare the capacities of the networks. Because the actual lost time per phase change (in seconds) is independent of the cycle length, a change in the signal cycle should affect only the fraction of cycle lost per phase change, l. Thus, the model can by applied to compare networks with different cycle lengths by using different values of l. For example, because two-way networks tend to have longer signal cycles than one-way networks, the value of l used for two-way networks should be smaller than the value of l used for one-way networks. From the results presented in Figures 3 and 4, a smaller value of l for two-way networks would yield capacities that are more competitive with one-way networks.

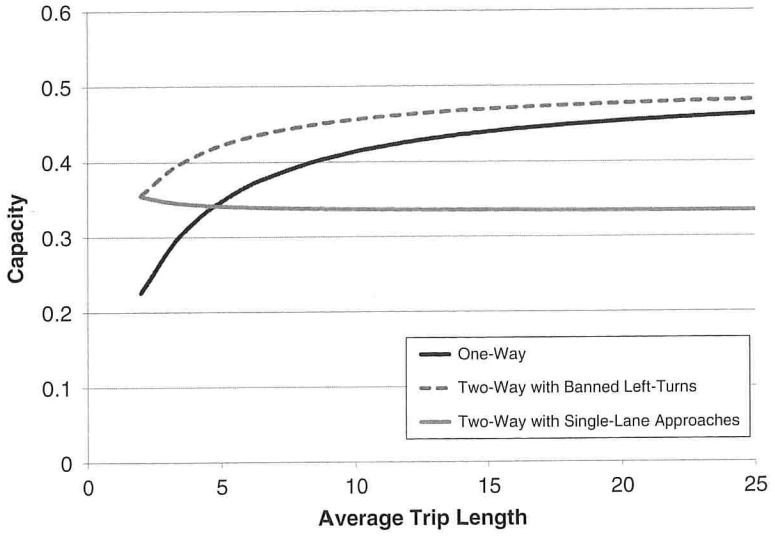
EXTENSIONS TO MORE REALISTIC NETWORKS

The previous results rely on some simplifying assumptions that may not occur on real networks. This section discusses how the results of this analysis may change when some assumptions are relaxed. These assumptions include the assumption that lost times and saturation flows are the same for left-turn and through movements, that left-turn phases are perfectly timed, and that traffic signals are evenly balanced between the north–south and east–west directions.

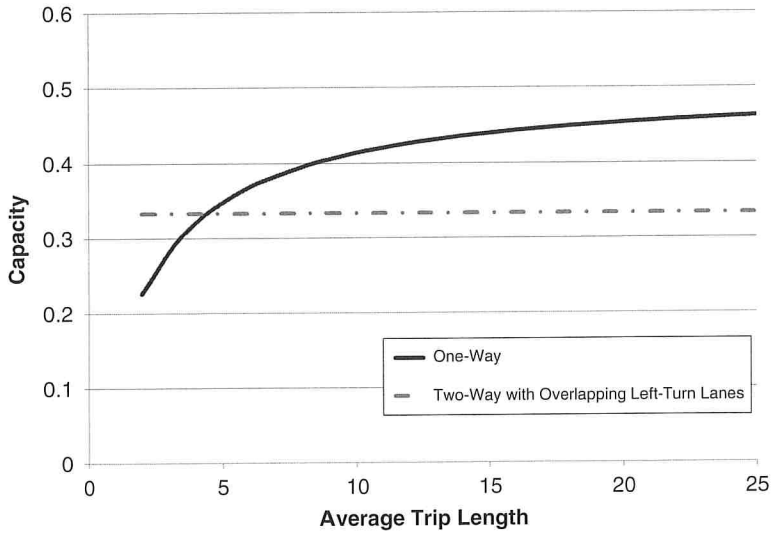
This analysis assumes equal lost times for all phase changes. At real intersections, the lost time incurred depends on the movements at the intersection. For example, work by Bonneson (*25*) shows that the lost times incurred by left-turning movements are greater than those incurred by through movements. This finding can easily be incorporated into the analysis by changing the fractions of the cycle available to serve each direction as calculated in Equations 2 and 6.

It was also assumed here that through and left-turning vehicles discharge at the same rate. From the *Highway Capacity Manual,* the saturation flow for left-turning vehicles in exclusive lanes is at most 0.95 times that for through vehicles (*24*). This finding can also be easily incorporated into the capacity calculations by modifying Equation 1 so that a different saturation flow ratio term is used for left-turn and through vehicles. In this case, Equation 1 would change to

$$C_i = \left[\frac{N_i^T}{N_i} g_i^T s_i^T + \frac{N_i^L}{N_i} g_i^L s_i^L \right] \frac{1}{\alpha_i} \tag{11}$$



(a)



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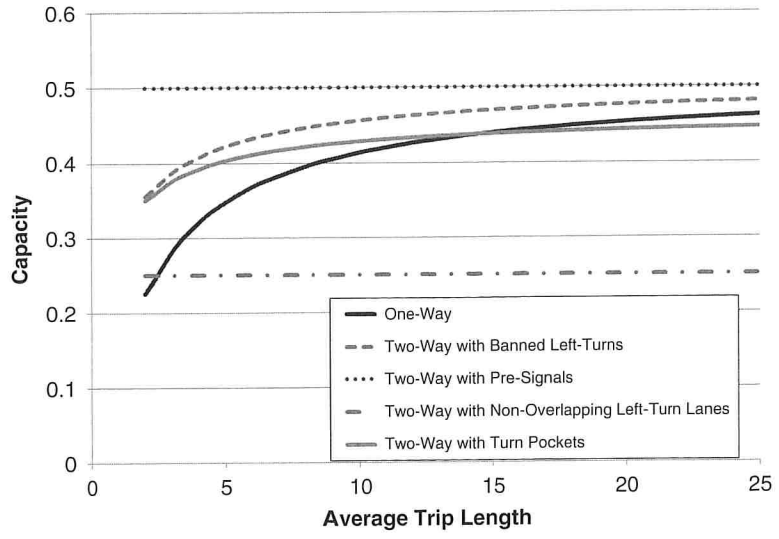
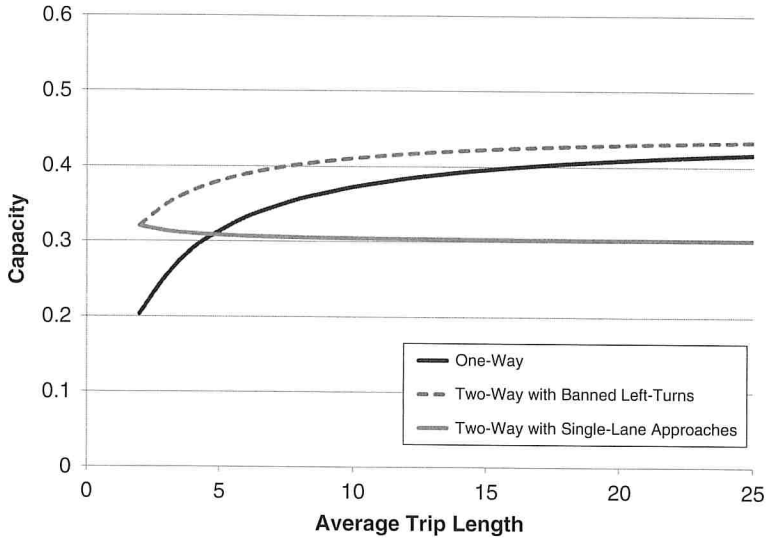
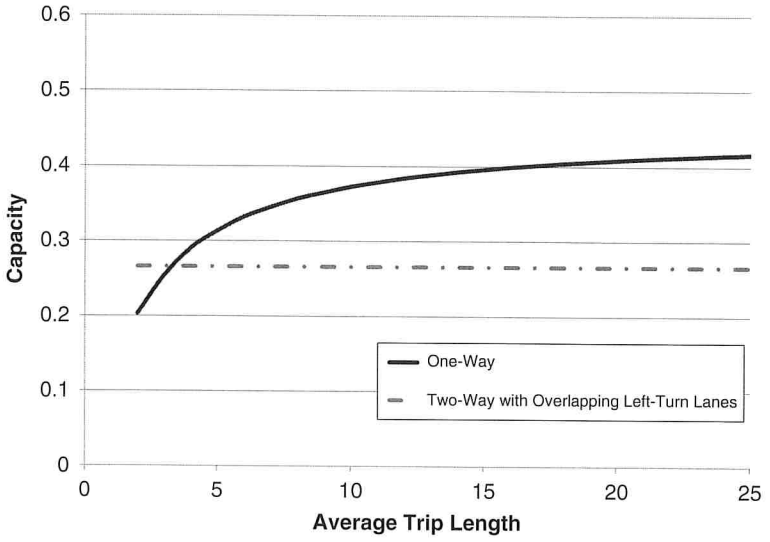


(c)

FIGURE 3 Capacities when $I = 0$ for (*a*) two-lane networks ($N_i = 2$), (*b*) three-lane networks ($N_i = 3$), and (*c*) four-lane networks ($N_i = 4$).



(a)



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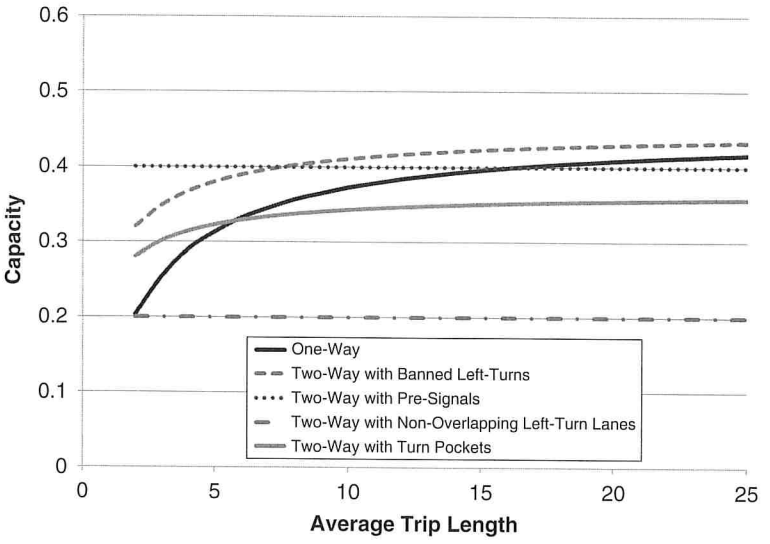

(c)

FIGURE 4 Capacities when $I = 0.05$ for (*a*) two-lane networks ($N_i = 2$), (*b*) three-lane networks ($N_i = 3$), and (*c*) four-lane networks ($N_i = 4$).

where s_i^T and s_i^L are the saturation ratio for through and left-turn vehicles, respectively. Both of these changes will reduce the capacity of networks that allow left turns. Although this modification would not change the general conclusion that two-way networks are able to provide higher capacities than one-way networks when trips are short and lost times are small, it would reduce the critical trip length at which one-way networks provide higher capacities.

This analysis also assumes the left-turn phases are perfectly timed to serve the left-turn demand. In reality, traffic engineers use a minimum left-turn phase length (usually about 7 to 10 s) to accommodate variation in the number of left turns per cycle. Thus, the capacity here is overestimated for situations in which the perfectly calculated left-turn phase length is shorter than this minimum length. However, the capacity analysis can easily be modified. Denoting the minimum green fraction required for the left-turn phase g_i^{L*}, Equation 11 can be changed to

$$C_i = \left[\frac{N_i^T}{N_i} g_i^T s_i^T + \frac{N_i^L}{N_i} g_i^L s_i^L\right]\frac{1}{\alpha_i} \quad \text{if } g_i^L \geq g_i^{L*}$$
$$C_i = \left[\frac{N_i^T}{N_i} g_i^T s_i^T + \frac{N_i^L}{N_i} x g_i^{L'} s_i^L\right]\frac{1}{\alpha_i} \quad \text{otherwise} \tag{12}$$

where x is the portion of the minimum left-turn phase used by vehicles to discharge on average. Equation 4 shows that g_i^L decreases with n. Thus, when minimum green times for the left-turn phase are included, the capacity of two-way networks will be reduced when trip lengths are long (i.e., for cases in which one-way networks already provide capacity superior to two-way networks). Therefore, the previous results that two-way networks are more competitive with one-way networks when trip lengths are short should still apply.

It was also assumed that origins and destinations were evenly distributed across the network so that travel is evenly split between the north–south and east–west directions. However, in some networks, the distribution of origins and destinations may be such that travel in one direction is favored over the other. For example, vehicles may travel more in the north–south direction than the east–west direction, on average. In this case, it would not make sense to provide the same green time to each direction. Instead, the direction with more demand should receive more green time. Analysis shows that if the green time is proportioned correctly, the capacities given by Equations 6a and 6b and presented in Figures 3 and 4 will not change if demand is unbalanced. Thus, the previous results still hold if signal timings are optimally adjusted to accommodate the different demand patterns.

SUMMARY AND CONCLUSIONS

This paper develops an analytical model to quantify and compare the trip-serving capacities of urban traffic networks. This capacity turns out to be a function of the number and types of lanes at each intersection, the lost time per signal phase change, and the average trip length. Planners and engineers may use this model as a tool to help determine whether to convert a one-way network into a two-way network.

The model unveils several general insights on the operation of urban traffic networks. Contrary to conventional wisdom and design handbooks, one-way networks are not always more efficient than two-way networks that allow left-turn movements. When average trip lengths are short, these two-way networks may be able to serve trips at a higher rater than one-way networks because the additional circuity in one-way networks offsets the more efficient intersection control. Two-way networks also become more competitive as the fraction of the signal cycle lost per phase change decreases. Because lost time is generally constant, two-way networks would be more competitive when signal cycles are increased.

This work also shows that two-way networks would always have higher trip-serving capacities than one-way networks if left turns are banned, even when average trip lengths are long. Although both strategies eliminate conflicting turning maneuvers so that intersections can be served with two-phase traffic signals, two-way networks with banned left turns impose less circuity on average than one-way networks. Thus, a one-way to two-way network conversion can always be performed while increasing the rate that trips can be served if left turns are simply banned at intersections.

These findings may be especially useful to urban planners seeking to increase the livability of downtown areas. Because one-way networks are undesirable because of faster vehicle speeds and increased user confusion, that they can be converted to two-way operation while serving trips at a higher rate means that both livability and efficiency objectives can be achieved simultaneously. Improved efficiency may also help increase the political acceptability of these conversions. In addition, the strategy of banning left turns at intersections of two-way streets is not as restrictive to emergency or transit vehicles as using one-way streets because turning restrictions at intersections can be lifted for these vehicle classes.

This work focuses only on the trip-serving capacity of the various networks by examining operation when the network is on the verge of oversaturation. However, most networks operate at peak efficiency for only a small portion of the time, and some do not approach capacity at all. Additional work is being performed to determine the best way to operate a downtown network when traffic densities are low and the network is operating below capacity. This work should yield insights on how best to serve trips on signalized networks when operating under light traffic conditions.

ACKNOWLEDGMENTS

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The Access Management Committee peer-reviewed this paper.

TRANSPORTATION RESEARCH RECORD:
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Peer Review Process

The ***Transportation Research Record: Journal of the Transportation Research Board*** publishes approximately 25% of the more than 4,000 papers that are peer reviewed each year. The mission of the Transportation Research Board (TRB) is to disseminate research results to the transportation community. The Record series contains applied and theoretical research results as well as papers on research implementation.

The TRB peer review process for the publication of papers allows a minimum of 30 days for initial review and 60 days for rereview, if needed, to ensure that only the highest-quality papers are published. A minimum of three reviews are required for a publication recommendation. The process also allows for scholarly discussion of any paper scheduled for publication, along with an author-prepared closure.

The basic elements of the rigorous peer review of papers submitted to TRB for publication are described below.

Paper Submittal: June 1–August 1

Papers may be submitted to TRB at any time. However, most authors use the TRB web-based electronic submission process available between June 1 and August 1, for publication in the following year's Record series.

Initial Review: August 15–November 15

TRB staff assigns each paper by technical content to a committee that administers the peer review. The committee chair assigns at least three knowledgeable reviewers to each paper. The initial review is completed by mid-September.

By October 1, committee chairs make a preliminary recommendation, placing each paper in one of the following categories:

1. Publish as submitted or with minor revisions;
2. Reconsider for publication, pending author changes and review; or
3. Reject for publication.

By late October, TRB communicates the results of the initial review to the corresponding author. Corresponding authors communicate the information to coauthors. Authors of papers in Category 2 (above) must submit a revised version addressing all reviewer comments, along with an explanation of how the comments have been addressed.

Rereview: November 20–January 25

The committee chair reviews revised papers in Category 1 (above) to ensure that the changes are made and sends the Category 2 revised papers to the initial reviewers for rereview. After rereview, the chair makes the final recommendation on papers in Categories 1 and 2. If the paper has been revised to the committee's satisfaction and ranks among the best papers, the chair may recommend publication. The chair communicates the results of the rereview to the authors.

Discussions and Closures: February 1–May 15

Discussions may be submitted for papers that will be published. TRB policy is to publish the paper, the discussion, and the author's closure in the same Record.

Many papers considered for publication in the *Transportation Research Record* are also considered for presentation at TRB meetings. Individuals interested in submitting a discussion of any paper presented at a TRB meeting must notify TRB no later than February 1. If the paper has been recommended for publication in the *Transportation Research Record*, the discussion must be submitted to TRB no later than April 15. A copy of this communication is sent to the author and the committee chair.

The committee chair reviews the discussion for appropriateness and asks the author to prepare a closure to be submitted to TRB by May 15. The committee chair reviews the closure for appropriateness. After the committee chair approves both discussion and closure, the paper, the discussion, and the closure are included for publication together in the same Record.

Final Manuscript Submittal: March 15

In early February, TRB requests a final manuscript for publication—to be submitted by March 15—or informs the author that the paper has not been accepted for publication. All accepted papers are published by December 31.

Paper Awards: April to January

The TRB Executive Committee has authorized annual awards sponsored by Groups in the Technical Activities Division for outstanding published papers:

- Charley V. Wootan Award (Policy and Organization Group);
- Pyke Johnson Award (Planning and Environment Group);
- K. B. Woods Award (Design and Construction Group);
- D. Grant Mickle Award (Operations and Preservation Group);
- John C. Vance Award (Legal Resources Group);
- Patricia F. Waller Award (Safety and System Users Group); and
- William W. Millar Award (Public Transportation Group).

Other Groups also may nominate published papers for any of the awards above. In addition, each Group may present a Fred Burggraf Award to authors 35 years of age or younger.

Peer reviewers are asked to identify papers worthy of award consideration. Each Group reviews all papers nominated for awards and makes a recommendation to TRB by September 1. TRB notifies winners of the awards, which are presented at the following TRB Annual Meeting.

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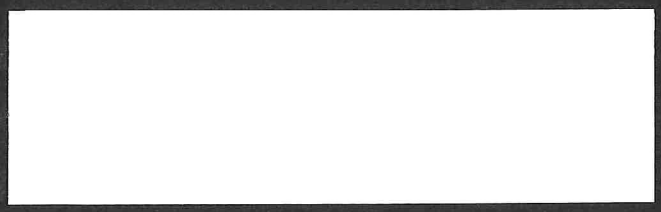

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